# Workshop Agenda

VIII. Load Rating Example #2
- Simple Span Nail Laminated Timber Deck (without distress)

IX. Class Exercise
- Simple Span Nail Laminated Timber Deck (with distress)

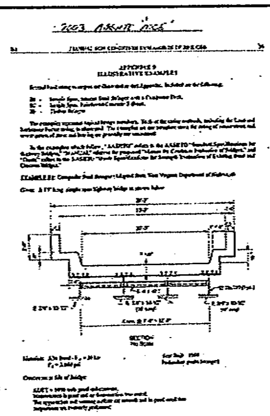
X. Review of worked out examples

XI. Submittal to MN/DOT and Review of Process

XII. Common mistakes and questions

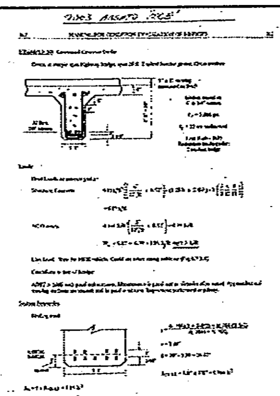
# Other Examples

- Simple Span, Interior Steel Stringer with Composite Deck



## Other Examples

- Simple Span, Reinforced Concrete T-Beam



## Other Examples

- ASR, Timber Abutment Pile with No Decay

MINNESOTA DEPARTMENT OF TRANSPORTATION

**ASR TIMBER ABUTMENT PILE RATING WORKSHEET**

[illegible]

Location: [illegible]

Given Information: - A simple span [illegible] bridge, [illegible]. Timber piles in [illegible] condition.
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- [illegible]
- Timber species: Douglas fir-larch, (coastal region)

Unit Definitions

[illegible]

Objective: [illegible] the 12" diam. timber abutment piles [illegible]

Reference MnDOT Spec. [illegible]

Input

Reference AASHTO Table [illegible]

Species: Douglas fir-larch (coastal region)

Commercial Grade: Dense Select Structural

Size Class: Posts

[illegible]

# Other Examples

- ASR, Timber Abutment Pile with Decay

MINNESOTA DEPARTMENT OF TRANSPORTATION

**ASR TIMBER ABUTMENT PILE RATING WORKSHEET**

[illegible]

Location: [illegible]

Given Information - A simple span [illegible] bridge, [illegible].
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- [illegible]
- Timber species: Douglas fir-larch, (coastal region)
- Piles are found to have internal decay with pile section loss of [illegible]

Unit Definitions

[illegible]

Objective: [illegible] the 12" diam. timber abutment piles [illegible]

Reference MnDOT [illegible]

Input

Reference AASHTO Table [illegible]

Species: Douglas fir-larch (coastal region)

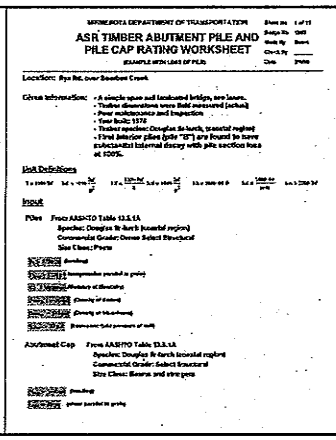
Commercial Grade: Dense Select Structural

Size Class: Posts

[illegible]

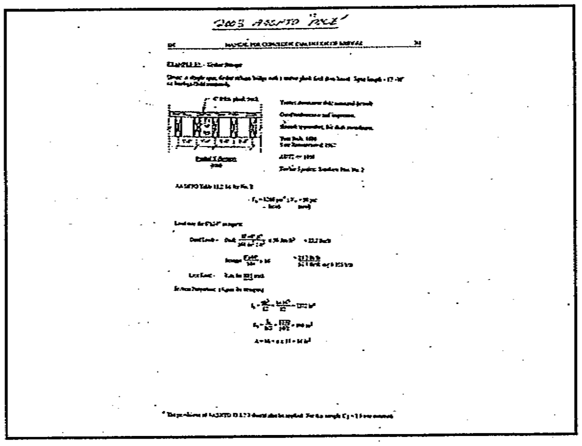
## Other Examples

- ASR, Timber Abutment Cap with Loss of Pile



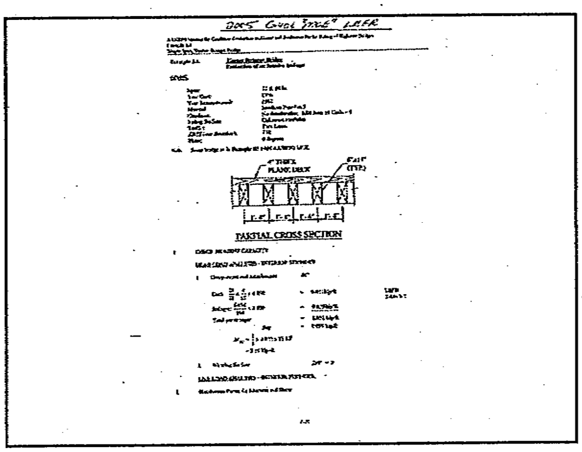

## Other Examples

- ASR, Simple Span Timber Stringer



# Other Examples

- LRFR, Simple Span Timber Stringer

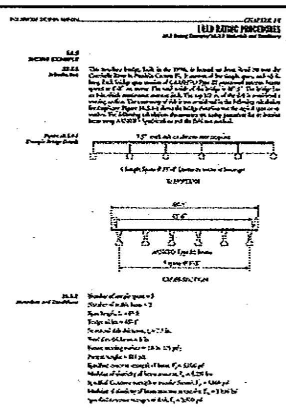


## Other Examples

- ASR (Using Old Forms)
  - Timber Beam
  - Timber Longitudinal Nailed Panel
  - Timber Longitudinal Glulam Deck
  - Timber Transverse Plank Deck

## Other Examples

- LFR & LRFR, Simple Prestressed Concrete Beams



2003 AASHTO "MCE"



# APPENDIX B
# ILLUSTRATIVE EXAMPLES

Several load rating examples are illustrated in this Appendix. Included are the following:

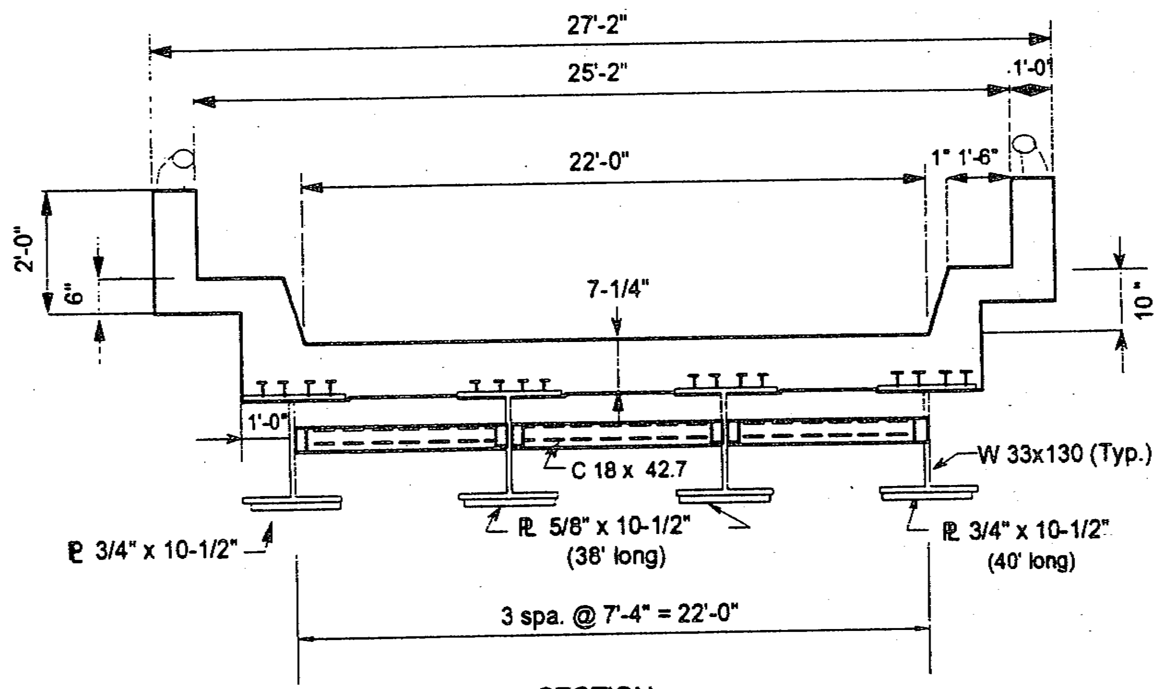
B1 - Simple Span, Interior Steel Stringer with a Composite Deck.
B2 - Simple Span, Reinforced Concrete T-Beam.
B3 - Timber Stringer

The examples represent typical bridge members. Each of the rating methods, including the Load and Resistance Factor rating, is illustrated. The examples are not complete since the rating of connections and investigation of shear and bearing are generally not considered.

In the examples which follow, "AASHTO" refers to the AASHTO "Standard Specifications for Highway Bridges," "MANUAL" refers to the proposed "Manual for Condition Evaluation of Bridges," and "Guide" refers to the AASHTO "Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges."

EXAMPLE B1: Composite Steel Stringer (Adapted from West Virginia Department of Highways)

Given: A 65' long, simple span highway bridge as shown below.




SECTION
No Scale

Materials: A36 Steel - $F_y$ = 36 ksi
$f'_c$ = 3,000 psi

Year Built : 1964
Redundant (multi-Stringer)

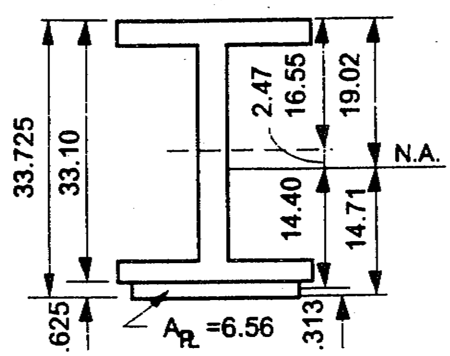
Conditions at Site of Bridge:

ADTT > 1000 with good enforcement.
Maintenance is good and no deterioration was noted.
The approaches and wearing surface are smooth and in good condition.
Inspections are routinely performed.

Rate a typical Interior Stringer

Section Properties: In unshored construction, the steel stringer must support its own weight plus the weight of the concrete slab. For the composite section, the concrete is transformed into an equivalent area of steel by dividing the area of the slab by the modular ratio. Live load plus impact stresses are carried by the composite section using a modular ratio of n. To account for the effect of creep, superimposed dead load stresses are carried by the composite section using a modular ratio of 3n. (AASHTO 10.38.1). The as-built section properties are used in this analysis.

Noncomposite: W33 x 130 & ℄ 5/8" x 10-1/2"
$t_f = 0.855"$; $b_f = 11.51"$; $t_w = 0.58"$
$A = 38.26\ in^2$




$$\bar{y} = \frac{(17.175)^{W}(38.26) + (.313)^{PL}(6.56)}{38.26 + 6.56}$$

$$\bar{y} = 14.71"$$

$$I_x = \overset{W}{6699} + \overset{W}{38.26(2.47)^2} + \overset{PL}{6.56(14.40)^2}$$
$$= 8293\ in^4$$

$$S_t = \frac{8293}{19.02} = 436.0\ in^3 = S_t^{DL}$$

$$S_b = \frac{8293}{14.71} = 563.7\ in^3 = S_b^{DL}$$

Composite Section Properties:

Effective Flange Width: (AASHTO 10.38.3.1)

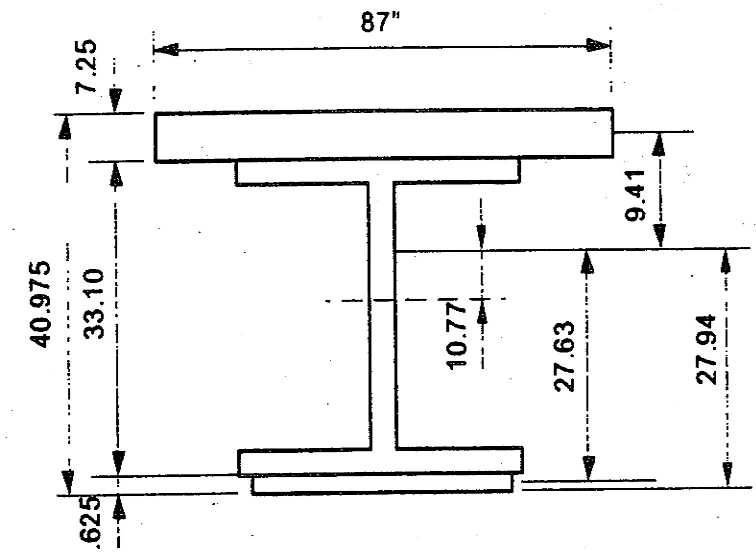
1/4(65)(12) = 195"
(7.33)(12) = 88"
(7.25)(12) = 87" ⇐ Controls

Modular Ratio (n): (MANUAL 6.6.2.4)
for $f'_c$ = 3,000 psi - n = 10

Composite Section Properties cont.:

Typical Interior Stringer:

Composite n = n: W33 x 130, ℄ 5/8" x 10-1/2" & Conc. 7-1/4" x 87"




$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{℄}{(.313)(6.56)} + \overset{Conc.}{(87 \times 7.25 \div 10)(37.35)}}{38.26 + 6.56 + (87 \times 7.25) \div 10}$$

$$\bar{y} = 27.94"$$

$$I_x = \overset{W}{6699} + \overset{W}{(38.26)(10.77)^2} + \overset{℄}{(6.56)(27.63)^2} + \overset{Conc.}{\frac{(87 \div 10)(7.25)^3}{12}} + \overset{Conc.}{(87 \times 7.25) \div 10\ (9.41)^2}$$

$$= 22007 \text{ in}^4$$

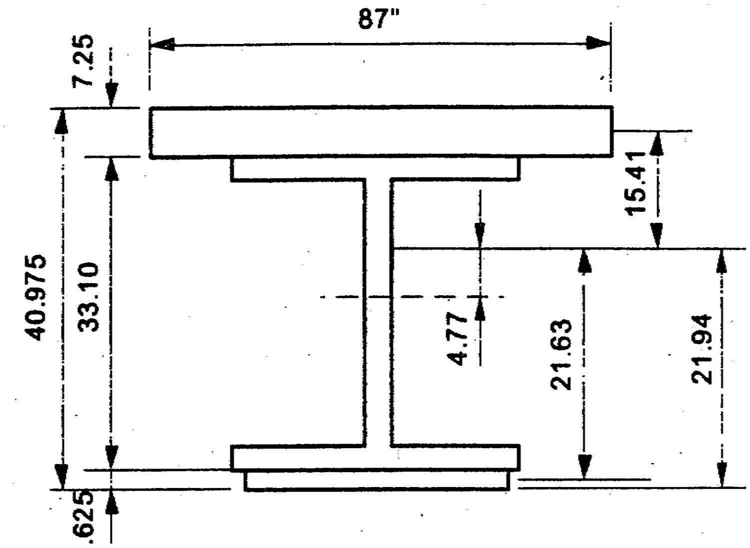
$$S_t = \frac{22007}{5.79} = 3801 \text{ in}^3$$ Section modulus at top of steel

$$S_b = \frac{22007}{27.94} = 787.7 \text{ in}^3 = S_b^L$$

use with Live Load

Composite Section Properties cont.:

Composite n = 3n: W33 x 130, ℄ 5/8" x 10-1/2" & Conc. 7-1/4" x 87"




$$\bar{y} = \frac{\overset{W}{(17.175)(38.26)} + \overset{℄}{(.313)(6.56)} + \overset{Conc.}{(87 \times 7.25 \div 30)(37.35)}}{38.26 + 6.56 + (87 \times 7.25) \div 30}$$

$$\bar{y} = 21.94"$$

$$I_x = \overset{W}{6699} + \overset{W}{(38.26)(4.77)^2} + \overset{℄}{(6.56)(21.63)^2} + \overset{Conc.}{\frac{(87 \div 30)(7.25)^3}{12}} + \left(\frac{87 \times 7.25}{30}\right)(15.41)^2$$

$$I_x = 15{,}725 \text{ in}^4$$

$$S_t = \frac{15725}{11.79} = 1333.8 \text{ in}^3$$ (Section modulus at top of steel)

$$S_b = \frac{15725}{21.94} = 716.7 \text{ in}^3 = S_b^{SDL}$$

use with Superimposed Dead Load (SDL)

Loads:

Dead Loads (includes an allowance of 6% of steel weight for connections):

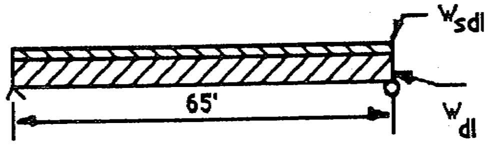
| | | |
|---|---|---|
| Deck $(7.33)\left(\frac{7.25}{12}\right)(150\text{ pcf})$ | = | 664.3 lbs/ft |
| Stringer (130)(1.06) | = | 137.8 lbs/ft |
| Cover ℄ (.625)(10.5)(490/144)(1.06)(38) ÷ 65 | = | 13.8 lbs/ft |
| Diaphragms (3)(42.7)(7.33)(1.06) ÷ 65 | = | 15.4 lbs/ft |
| Total per stringer | | 831.3 lbs/ft |

Superimposed Dead Loads: (see AASHTO 3.23.2.3.1.1)

| | | |
|---|---|---|
| Curb $(1)\left(\frac{10}{12}\right)(150\text{ pcf}) \div 2$ | = | 62.5 lbs/ft |
| Parapet $\left[\left(\frac{6 \times 19}{144}\right) + \left(\frac{18 \times 12}{144}\right)\right](150\text{ pcf}) \div 2$ | = | 171.9 lbs/ft |
| Railing (assume 20 plf) ÷ 2 | = | 10.0 lbs/ft |
| Wearing Surface | = | 0.0 |
| Total per stringer | | 244.4 lbs/ft |

Live Load: Rate for HS20

Moments:




$W_{sdl} = 0.244$ k/ft $\qquad M_{DL} = \frac{w_{dl} L^2}{8} = \frac{.831(65)^2}{8} = 439$ ft-k

$W_{dl} = 0.831$ k/ft $\qquad M_{SDL} = \frac{w_{sdl} L^2}{8} = \frac{.244(65)^2}{8} = 129$ ft-k

$M_L$ - From MANUAL, Appendix A3, page 74(1)

| Span | $M_L$ |
|---|---|
| 60 | 403.3 |
| 70 | 492.8 |

⇐ 65'

$$M_L = \frac{403.3 + 492.8}{2}$$

$M_L \doteq 448$ ft-k

(without Impact, without Dist.)

(1) Note the moments given in MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUALvalue.

Allowable Stress Rating (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(Consider Maximum Moment Section only for this example - See general notes.)

Impact - MANUAL 6.7.4. Use standard AASHTO

AASHTO 3.8.3.1

$$I = \frac{50}{L+125} \leq 0.3$$

$$I = \frac{50}{65+125} = 0.26$$

Distribution - MANUAL 6.7.3 indicates that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1

$$DF = \frac{S_s}{5.5} = \frac{7.33^{FT}}{5.5} = 1.33$$

Thus:

$$M_{L+I} = M_L\,(1+I) * DF = 448(1+0.26)(1.33)$$

$$M_{L+I} = 751 \text{ ft-k}$$

Inventory Level: MANUAL 6.6.2.1, Table 6.6.2.1-1 (bottom steel in tension controls)

For steel with $F_y = 36$ ksi $\rightarrow f_I = 0.55\ f_y$

Thus:

$$f_I = 0.55(36) = 20 \text{ ksi}$$

The Resisting Capacity $(M_{RI}) = f_I\ S_x^L$

$$M_{RI} = 20 \text{ ksi } (787.7 \text{ in.})^3 = 15754 \text{ in-k} = 1313 \text{ ft-k}$$

Then:

$$RF_I = \frac{M_{RI} - M_{DL}\dfrac{S_b^L}{S_b^{DL}} - M_{SDL}\dfrac{S_b^L}{S_b^{SDL}}}{M_{L+I}}$$

$$= \frac{1313 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{557.8}{751}$$

$$= \underline{\underline{0.74}} \text{ or } 0.74 \times 36 \text{ tons} = \underline{\underline{26.7 \text{ tons}}}$$

Alternatively, in terms of stress:

$$RF_I = \frac{f_s - \frac{M_{DL}}{S_b^{DL}} - \frac{M_{SDL}}{S_b^{SDL}}}{\frac{M_{L+I}}{S_b^{L+I}}}$$

$$= \frac{20\text{ ksi} - \frac{439^{\text{ft-k}} \text{ x } 12^{\text{in/ft}}}{563.7^{\text{in}^3}} - \frac{129^{\text{ft-k}} \text{ x } 12^{\text{in/ft}}}{716.7^{\text{in}^3}}}{\frac{751^{\text{ft-k}} \text{ x } 12^{\text{in/ft}}}{787.7^{\text{in}^3}}}$$

$$= \frac{20 - 9.345 - 2.160}{11.441}$$

$$= \frac{8.495}{11.441} = 0.74 \text{ as above}$$

Operating Level: MANUAL 6.6.2.1, Table 6.6.2.1-2

For steel with $F_y = 36$ ksi $\rightarrow f_o = 0.75\ f_y$

Thus:

$$f_o = 0.75(36) = 27 \text{ ksi}$$

and

$$M_{RO} = 27(787.7) = 21268 \text{ in-k} = 1772 \text{ ft-k}$$

and

$$R_{FO} = \frac{1772 - 439\dfrac{787.7}{563.7} - 129\dfrac{787.7}{716.7}}{751} = \frac{1016.8}{751}$$

$$RF_O = \underline{\underline{1.35}} \quad \text{or} \quad 1.35 \times 36 \text{ tons} = \underline{\underline{48.7 \text{ tons}}}$$

Load Factor Rating: MANUAL 6.4.2, 6.5.3 & 6.6.3

(Consider maximum moment section only for this example - see General Notes.)

Impact - MANUAL 6.7.4 – use standard AASHTO

From AS Rating I = 0.26

Distribution – MANUAL 6.7.3 – use standard AASHTO

From AS Rating DF = 1.33

$$M_{L+I} = M_L(1+I)\,DF = 448(1+0.26)(1.33)$$

$$= 751 \text{ ft-k (as for AS Rating)}$$

Capacity of Section: ($M_R$) – MANUAL 6.6.3.1

For braced, compact, composite sections:

$$M_R = M_u \text{ (AASHTO 10.50.1.1)}$$

where $M_u$ is found in accordance with applicable load factor provisions of AASHTO.

Check assumptions:

(1) Section is fully braced along top flange by composite deck (for Live Load & SDL)

(2) To check if section is compact, need to apply provisions of AASHTO 10.50.1.1.2. These checks follow.

Eqn: 10-123: $C_{CONC} = 0.85 f_c\, b_{eff}\, t_s = 0.85(3\,\text{ksi})(87\,\text{in})(7.25\,\text{in}) = 1608\,\text{k}^*$

Eqn: 10-124: $C_{STL} = A_s f_y = (38.26\,\text{in}^2 + 6.56\,\text{in}^2)(36\,\text{ksi}) = 1613.5\,\text{k}$

$C_{CONC} < C_{STL} \therefore C_{CONC} = 1608$ controls (10.50.1.1.1(a))

Capacity – per AASHTO 10.50.1.1.1(c)

Eqn: 10-126: $C' = \dfrac{\Sigma\,(AF_y) - C}{2} = \dfrac{1613.5 - 1608}{2} = 2.75\,\text{k}$

and applying AASHTO 10.50.1.1.1(d)

$(AF_y)_{TF} = (11.51 \times .855)(36) = 354\,\text{k} >>> 2.75\text{k} \therefore$ NA in top flange

Eqn: 10-127: $\bar{y} = \dfrac{C'}{(AF_y)_{TF}}\, t_{TF} = \dfrac{2.75}{354}(.855) = 0.007$ in neglect. Say NA at top of steel.

Since the PNA is at the top of the top flange, the depth of the web in compression at the plastic moment, $D_{cp}$ is equal to zero. Hence, the web slenderness requirement given by Equation (10-129) in Article 10.50.1.1.2 is automatically satisfied.

INTERIM 2001

* Neglects reinforcement in slab.

Check the ductility requirement given by Equation (10-129a) in Article 10.50.1.1.2:

Eqn: 10-129a $\left(\frac{D_p}{D'}\right) \le 5$

$$D' = \beta \frac{(d + t_s + t_h)}{7.5} \qquad \beta = 0.9 \text{ for } F_y = 36{,}000 \text{ psi}$$

$$D' = 0.9 \frac{(33.725 + 7.25 + 0.0)}{7.5} = 4.92$$

$$D_p = 7.25 \text{ in}$$

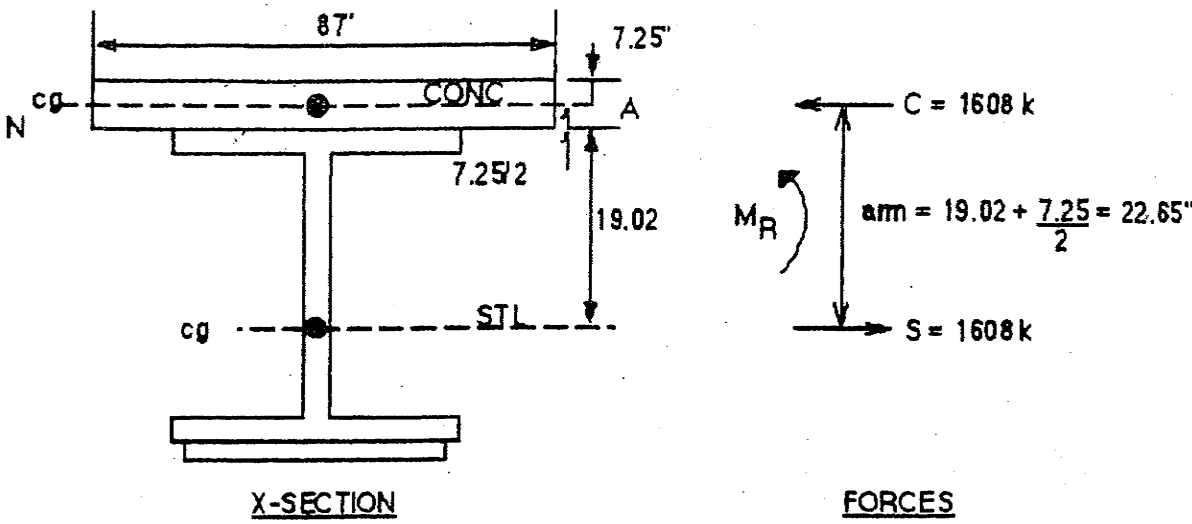
$$\left(\frac{D_p}{D'}\right) = \frac{7.25}{4.92} = 1.47 < 5 \quad \text{ok}$$

Since the top flange is braced by the hardened concrete deck, local and lateral buckling requirements need not be checked. The capacity of composite beams in simple spans satisfying the preceding web slenderness and ductility requirements is given by Equation (10-129c) in Article 10.50.1.1.2 when $D_p$ exceeds D':

$$C = M_R = \frac{5M_p - 0.85M_y}{4} + \frac{0.85M_y - M_p}{4}\left(\frac{D_p}{D'}\right)$$

$$M_y = F_y S = (36)\frac{787.7}{12} = 2363 \text{ ft-k}$$

Compute the plastic moment capacity $M_p$.




$M_p = C * \text{arm} = S * \text{arm} = 1608(22.65) = 36421$ in-k $= 3035$ ft-k

$$M_R = \frac{5(3035) - 0.85(2363)}{4} + \frac{0.85(2363) - 3035}{4}(1.47)$$

$$= 2914 \text{ ft-k}$$

Inventory Level: MANUAL 6.5.1 and 6.6.3

$$RF_I^{L+I} = \frac{M_R - A_1 M_D}{A_2 M_{L+I}} \quad \text{(MANUAL Eqn: 6-1a)}$$

where: (MANUAL 6.5.3)

$A_1 = 1.3$
$A_2 = 2.17$

Thus:
$$RF_I^{LF} = \frac{(2914) - 1.3(439 + 129)}{2.17(751)}$$

$$RF_I^{LF} = \underline{\underline{1.33}} \text{ or } 1.33 \text{ x } 36 \text{ tons} = \underline{\underline{47.9 \text{ tons}}}$$

Operating Level: MANUAL 6.5.3

Only change is $A_2 = 1.3$

Thus:
$$RF_O^{LF} = \frac{2.17}{1.3} RF_I^{LF} = \frac{2.17}{1.3}(1.33)$$

$$RF_O^{LF} = \underline{\underline{2.22}} \text{ or } 2.22 \text{ x } 36 \text{ tons} = \underline{\underline{79.9 \text{ tons}}}$$

Check Serviceability Criteria – AASHTO 10.57.2

At Inventory Level (bottom steel in tension controls)

$$f_D + 1.67\,(RF_I^{LF}) f_{L+I} \leq \text{Serv. Strength} = 0.95\,F_y$$

$$RF_I^{LF} = \frac{0.95\,F_y - f_D - f_{SDL}}{1.67\,f_{L+I}}$$

$$= \frac{0.95\,(36\text{ ksi}) - \dfrac{439(12)}{563.7} - \dfrac{129(12)}{716.7}}{1.67\,\dfrac{751(12)}{787.7}}$$

$$RF_I^{LF} = \underline{\underline{1.19}} \text{ or } 1.19 \text{ x } 36 \text{ tons} = \underline{\underline{42.8 \text{ tons}}}$$

Check the web compressive stress:

Eqn: 10-173

$$C = F_{cr} = \frac{26{,}200{,}000\,\alpha\,k}{\left(\dfrac{D}{t_w}\right)^2} \leq F_{yw}$$

where:
$$k = 9(D/D_c)^2$$
$$\alpha = 1.3$$

Since $D_c$ is a function of the dead-to-live load stress ratio according to the provisions of Article 10.50(b), an iterative procedure may be necessary to determine the rating factor:

Compute the compressive stresses at the top of the web:

$$f_D = \frac{439(12)(18.165)}{8293} = 11.5 \text{ ksi}$$

$$f_{SDL} = \frac{129(12)(10.935)}{15,725} = 1.1 \text{ ksi}$$

$$f_{L+I} = \frac{(1.67)(751)(12)(4.935)}{22,007} = 3.4 \text{ ksi}$$

$$\sum = 16.0 \text{ ksi}$$

Compute the tensile stresses at the bottom of the web:

$$f_D = \frac{439(12)(13.23)}{8293} = 8.4 \text{ ksi}$$

$$f_{SDL} = \frac{129(12)(20.46)}{15,725} = 2.0 \text{ ksi}$$

$$f_{L+I} = \frac{(1.67)(751)(12)(26.46)}{22,007} = 18.1 \text{ ksi}$$

$$\sum = 28.5 \text{ ksi}$$

$$D_c = 31.39\left(\frac{16.0}{16.0+28.5}\right) = 11.29''$$

$$k = 9(D/D_c)^2 = 9(31.39/11.29)^2 = 69.9$$

$$C = F_{cr} = \frac{26,200,000(1.3)(69.6)}{\left(\frac{31.39}{0.58}\right)^2(1000)} = 809.3 \text{ ksi} > F_{yw}$$

$$\therefore F_{cr} = F_{yw} = 36 \text{ ksi}$$

$$RF_I^{LF} = \frac{36-11.5-1.1}{3.4} = \underline{\underline{6.9}} \text{ or } (6.9)(36 \text{ tons}) = \underline{\underline{248.4 \text{ tons}}}$$

Since the computed rating factor would cause the total stresses in the tension flange to far exceed $F_y$ (causing the neutral axis to be higher on the web), further iterations are not necessary in this case. The web compressive stress does not govern the serviceability rating.

At Operating Level:

$$f_D + RF_O^{LF}(f_{L+I}) \leq \text{Serv. Strength}$$

Thus: $$R_O^{LF} = RF_I^{LF} \times 1.67 = 1.19 \times 1.67$$

$$R_O^{LF} = \underline{\underline{1.98}} \text{ or } 1.98 \times 36 \text{ tons} = \underline{\underline{71.3 \text{ tons}}}$$

Load Factor Summary

| | RF | TONS | CONTROLLED |
|---|---|---|---|
| Inventory | 1.19 | 42.8 | Art. 10.57.2 |
| Operating | 1.98 | 71.3 | Art. 10.57.2 |

Load and Resistance Factor Rating: (See *AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*)

(Again consider maximum moment section only for this example – see General Notes.)

Impact - Guide 3.3.2.3 may vary based on condition of wearing surface. But for comparison purposes use same I as for AS method:

$$I = 0.26$$

Distribution - Guide 3.3.3 use standard AASHTO

$$DF = 1.33$$

Live Load – Guide 3.3.2.2 use HS20 to be consistent with other rating methods. Normally would use rating vehicles (Guide Fig. 2) or lane loading (Guide Fig 3).

Thus: $$M_{L+I} = M_R\ (1+I) \times DF = 448 \times (1+0.26)(1.33)$$

$$= 751 \text{ ft-k}$$

Capacity of Section – Guide 3.3.2.4

$M_R$ is based on AASHTO 10.50 as for Load Factor

Thus: $$M_R = 2914 \text{ ft-k (Point Page 98.1)}$$

$$RF^{LRF} = \frac{\phi M_R - \gamma_D M_D}{\gamma_L M_{L+I}} \quad \text{(Guide Eqn. 2)}$$

where:

| | | | |
|---|---|---|---|
| $\phi$ | = | 0.95 | (Guide 3.3.4.2, Table 3(b)) |
| $\gamma_D$ | = | 1.2 | (Guide, Table 2) |
| $\gamma_L$ | = | 1.45 | (Guide, Table 2) |

Load and Resistance Factor Rating (*continued*):

then:

$$RF^{LRF} = \frac{0.95(2914) - 1.2(439 + 129)}{1.45(751)}$$

$$RF^{LRF} = \underline{\underline{1.91}} \text{ or } 1.91 \text{ x } 36 \text{ tons} = \underline{\underline{68.6 \text{ tons}}}$$

SUMMARY OF RESULTS

| | RF | HS20 Truck | H20[1] Truck |
|---|---|---|---|
| Allowable Stress: | | | |
| Inventory | 0.74 | 26.7 | 21 |
| Operating | 1.35 | 48.7 | 38.3 |
| Load Factor: | | | |
| Inventory | 1.19 | 42.8 | 33.8 |
| Operating | 1.98 | 71.3 | 56.2 |
| Load and Resistance Factor | 1.91 | 68.8 | 54.2 |

(1)

$$H_{TR} = RF \text{ x } \frac{M_L^{HS20}}{M_L^{H20}} \text{ x } 20^T \qquad M_L^{HS20} = 448 \text{ (page 87)}$$

$$= RF\frac{448}{316} \text{ x } 20^T \qquad M_L^{H20} = \frac{20}{15} M_L^{H15} = \frac{20}{15}\,\frac{(265.1 + 209.2)}{2} = 316 \text{ ft-k}$$

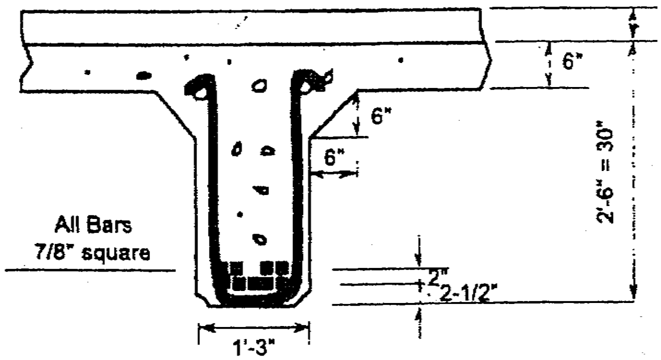
$$H_{TR} = RF \text{ x } (1.42) \text{ x } 20^T$$

2003 AASHTO MCE



EXAMPLE B2: Reinforced Concrete Girder

Given: A simple span highway bridge, span 26 ft. Typical interior girder. Cross section:




Girders spaced on
6'-6-1/4" centers

$f'_c = 3{,}000$ psi

$f_y = 33$ ksi (unknown)

Year Built - 1925
Redundant (multi-girder)
Two-lane bridge

Loads:

Dead Loads on interior girder:

Structural Concrete:

$$0.15\,\text{k/ft}^3\left[\left(\frac{6''}{12''/\text{ft}} \text{ x } 6.52'\right)+\left(1.25\,\text{ft x } 2.0\,\text{ft}\right)+2\left(\frac{1}{2}\frac{6}{12}\frac{6}{12}\right)\right]$$

$$= 0.87\,\text{k/ft}$$

AC Overlay:

$$0.144\ \text{k/ft}^3\left(\frac{5''}{12''/\text{ft}} \text{ x } 6.52'\right) = 0.39\ \text{k/ft}$$

$$W_{dL} = 0.87 + 0.39 = 1.26\ \text{k/ft}\ \ \underline{\underline{\text{say } 1.3\ \text{k/ft}}}$$

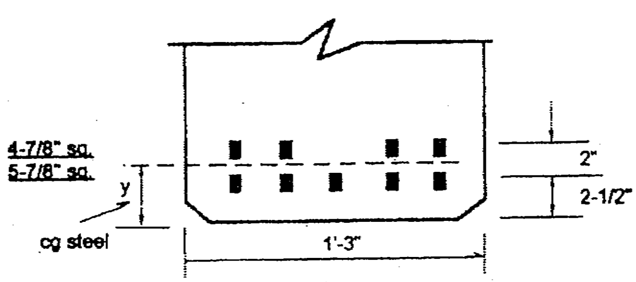
Live Load - Rate for HS20 vehicle. Could use other rating vehicles (Fig.6.7.2.4).

Conditions at Site of Bridge:

ADTT > 1000 with good enforcement. Maintenance is good and no deterioration noted. Approaches and wearing surfaces are smooth and in good condition. Inspections performed regularly.

Section Properties

Find cg steel




$$y = \frac{4(.766)(2 + 2\text{-}1/2) + 5(.766)(2\text{-}1/2)}{4(.766) + 5(.766)}$$

$y = 3.39''$

$d = 30'' - 3.39 = 26.61''$

$A_{1BAR} = 7/8'' \times 7/8'' = 0.766\ \text{in}^2$

$A_s = 9 \times A_{1BAR} = 6.89\ \text{in}^2$

Effective Slab Width (for T-Girder)

AASHTO 8.10.1.1

$$1/4\ L = \frac{26\text{ ft x }12\text{ in-ft}}{4} = 78"$$

or

$$\text{CC SPCG} = 6' - 6\text{-}1/4" = 78.25"$$

or

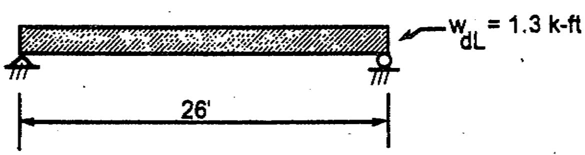
$$12\ t_s = 12 \times 6" = 72" \Leftarrow \text{Controls}$$

$$\rho_{act} = \frac{A_s}{b_{eff}\ d} = \frac{6.89}{72" \times 26.61} = 0.0036$$

(if compression within flange)

Midspan Moments:

Live Load - HS20




$$M_d = \frac{w_{dL}\ L^2}{8} = \frac{1.3\text{ k/ft x }26^2\text{ft}^2}{8} = 109.9\text{ k-ft}$$

For HS20 - From MANUAL, Appendix A3, page 74(1). Using Table, select from column "Without Impact"

$M_L$ = 111.1 k-ft (without impact and without distribution)

(1) Note the moments given in the MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUAL values.

Allowable Stress Rating (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(For this example we consider only the maximum moment section - see General Notes)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.2.1

$$I = \frac{50}{L + 125} \leq 0.30$$

$$I = \frac{50}{26 + 125} = 0.33 \text{ use } \underline{0.30}$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1

$$DF = \frac{S_G}{6.0} \quad \text{Concrete T-Beam}$$

$$DF = \frac{6' - 6\text{-}1/4''}{6.0} = \frac{6.52'}{6} = 1.087$$

Thus:

$$M_{L+I} = M_L\,(1+I)\,(DF) = 111.1(1 + .30)(1.087)$$

$$= 157 \text{ ft-k}$$

Inventory Level: MANUAL 6.5.2 & 6.6.2.4 - The inventory unit stresses are determined in accordance with AASHTO "Service Load Design Method" Article 8.15 or taken from MANUAL 6.6.2.4.

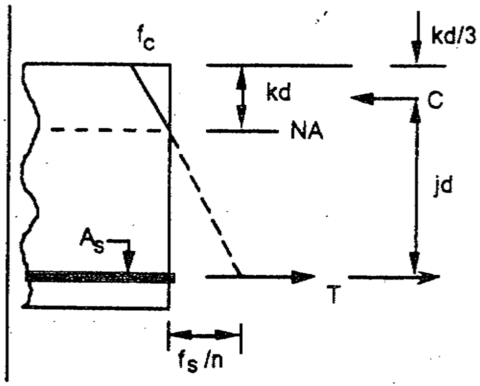
Thus Inventory allowable stresses, AASHTO 8.15.2.1.1

$$f_c^I = 0.4\,f'_c = 0.4\,(3000 \text{ psi}) = 1200 \text{ psi} = 1.2 \text{ ksi}$$

For Reinforcing Steel, MANUAL 6.6.2.3 controls

$$f_s^I = 18000 \text{ psi} = 18 \text{ ksi (unknown steel prior to 1954)}$$

Capacity (Traditional Approach):




Stress & Force Diagram
(nts)

The actual steel and concrete stresses are not known and must be found. Since this is a T-beam, assume neutral axis (na) is within slab. Thus, rectangular beam formulas apply. Check this assumption later.

Position of Neutral Axis:

$$k = \sqrt{2\ \rho n + (\rho n)^2} - \rho n$$

where: $\rho = \frac{As}{bd} = \frac{6.89\text{ in}^2}{(72\text{ in})(26.61\text{ in})}$

$\rho = 0.0036$

$n = \frac{E_s}{E_c}$

$n = 10$ (from Article 6.6.2.4)

$$k = \sqrt{2(.0036)(10) + ((.0036)(10))^2} - (.0036)(10)$$

$$k = 0.235$$

$$j = 1 - \frac{k}{3} = 1 - \frac{.235}{3} = 0.922$$

Then

Capacity if concrete allowable stress controls—

$$M_c = 1/2\ f_c\ jk\ bd^2$$

$$= 1/2\ (1.2\text{ ksi})(0.922)(0.235)(72\text{ in})(26.61\text{ in})^2$$

$$= 6622.8\text{ in-k} = 552\text{ ft-k}$$

Capacity if steel reinforcement allowable stress controls—

$$M_s = A_s\ f_s\ j\ d$$

$$M_s = (6.89\text{ in}^2)(18\text{ ksi})(0.922)(26.61\text{ in})$$

$$M_s = 3042.8\text{ in-k} = 253\text{ ft-k} \Leftarrow \text{Controls since } M_s < M_c.$$

Check neutral axis assumption:

$k_d = (0.235)(26.61 \text{ in}) = 6.25" > 6"$ the slab thickness $\therefore$ NA is below bottom of slab and slightly into web. This could be ignored in this case. However for the sake of completeness, capacity will be figured below based on the NA below the slab and ignoring the compression in the stem concrete.

$$kd = \frac{2nd\,A_s + bt^2}{2n\,A_s + 2bt}$$

$$kd = \frac{2(10)(26.61 \text{ in})(6.89 \text{ in}) + (72 \text{ in})(6 \text{ in}^2)}{2(10)(6.89 \text{ in}) + 2(72 \text{ in})(6 \text{ in})} = \frac{6258.9}{1001.8}$$

$$kd = 6.25 \text{ in} \rightarrow k = \frac{kd}{d} = \frac{6.25 \text{ in}}{26.61 \text{ in}} = 0.235$$

$$Z = \left(\frac{3kd - 2t}{2kd - t}\right)\frac{t}{3}$$

$$Z = \left(\frac{3(6.25 \text{ in}) - 2(6 \text{ in})}{2(6.25 \text{ in}) - (6 \text{ in})}\right)\frac{6 \text{ in}}{3} = \frac{6.75 \text{ in}}{6.5 \text{ in}}(2 \text{ in})$$

$$Z = 2.077 \text{ in.}$$

$$j\,d = d - Z$$

$$j\,d = 26.61 \text{ in} - 2.077 \text{ in} = 24.53 \text{ in}$$

$$M_s = A_s\,f_s\,j\,d$$

$$M_s = (6.89 \text{ in}^2)(18 \text{ ksi})(24.53 \text{ in}) = 3042.2 \text{ in-k}$$

$$M_s = 253 \text{ ft-k} \quad \text{as before}$$

(Note concrete was not checked since capacity of section is limited by steel allowable stress.)

$$RF_I^A = \frac{M_{RI} - M_D}{M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

$$RF_I^A = \frac{253 \text{ ft-k} - 109.9 \text{ ft-k}}{157 \text{ ft-k}} = 0.91$$

Operating Level: MANUAL 6.5.2 & 6.6.2.4

The operating allowable stresses, MANUAL 6.6.2.4 for $f'_c = 3{,}000$ psi:

$$f_c^O = 1900 \text{ psi} = 1.9 \text{ ksi}$$

For Reinforcing Steel, MANUAL 6.6.2.3 controls:

$$f_s^o = 25{,}000 \text{ psi} = 25 \text{ ksi (unknown steel, prior to 1954)}$$

The basic relationships defined previously apply:

Since ρ and n do not change, the neutral axis, k, j and Z terms do not change.

Thus:

$$M_S = A_S\, f_s\, j\, d$$

$$= (6.89 \text{ in}^2)(25 \text{ ksi})(24.53 \text{ in})$$

$$= 4225.3 \text{ in-k} = 352 \text{ ft-k}$$

and checking concrete stress to ensure that concrete does not control

$$f_c = \frac{f_s}{n}\left(\frac{k}{1 - k}\right)$$

$$f_c = \left(\frac{25 \text{ ksi}}{10}\right)\left(\frac{0.235}{1 - .235}\right) = 0.77 \text{ ksi} << 1.9 \text{ ksi allowable}$$

Therefore, capacity of section is controlled by allowable steel stress.

$M_{R_O} = 352$ ft-k

$$RF_O^A = \frac{M_{R_O} - M_D}{M_{L+I}} = \frac{352 \text{ ft-k} - 109.9 \text{ ft-k}}{157 \text{ ft-k}}$$

$RF_O^A = 1.54$

Load Capacity Based on Allowable Stress

Inventory: $0.91 \times 36^T = 32.8^T$ HS

Operating: $1.54 \times 36^T = 55.4^T$ HS

To transform "HS" rating to "H" rating multiply HS rating factor by ratio of "HS" moment to "H" moment:

For 26' span: $M_L^{HS20} = 111.1$ ft-k (see Sheet 97)

and using MANUAL Appendix A3, pg. 74 → $M_L^{H15} = 78$ ft-k

Then

$$M_L^{H20} = \frac{20^T}{15^T} \times 78 \text{ ft-k} = 104 \text{ ft-k}$$

and

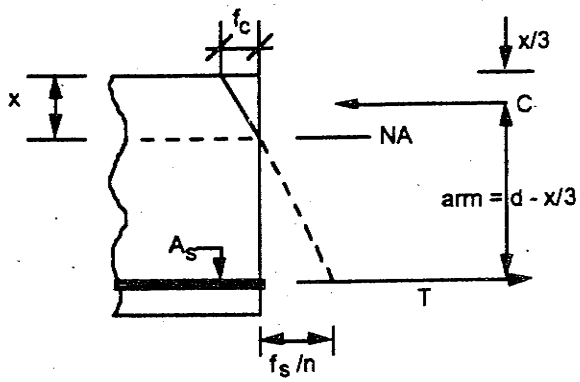
$$\text{Ratio} = \frac{M_L^{HS20}}{M_L^{H20}} = \frac{111.1}{104} = 1.068$$

Thus for H20 Truck:

Inventory: $0.91 \times 1.068 \times 20^{T} = 19.4^{T}$ H

Operating: $1.54 \times 1.068 \times 20^{T} = 32.9^{T}$ H

Capacity (Alternate Approach):




Since the location of the neutral axis (NA) and the corresponding stresses in the steel and concrete are not known, these must be determined consistent with the principles of equilibrium of the cross section.

Stress & Force Diagram
(nts)

(1) From the stresses on the cross section:

$$\frac{f_c}{x} = \frac{f_s/n}{d - x} \rightarrow f_c = \frac{f_s}{n}\left(\frac{x}{d - x}\right) \qquad \text{Eqn. 1}$$

(2) Assume the steel allowable stress controls the capacity of the section. This will be checked later. Then

$$T = A_s f_s = (6.89 \text{ in}^2)(18 \text{ ksi}) = 124 \text{ k}$$

and

$$C = 1/2\, f_c\, b\, x$$

but

$$C = T$$

thus,

$$1/2\, f_c\, b\, x = A_s f_s$$

$$x = \frac{A_s f_s}{1/2\, f_c\, b} \qquad \text{Eqn. 2}$$

Solve equations 1 and 2 to find location of neutral axis. This may be done by trail and error as follows.

Assume $f_s$ = 18 ksi, i.e. steel allowable stress controls.

Try x = 6.0 in. Then by Eqn. 1:

$$f_c = \frac{f_s}{n}\left(\frac{x}{d - x}\right) = \frac{18\text{ ksi}}{10}\left(\frac{6.0\text{ in}}{26.61\text{ in} - 6.0\text{ in}}\right) = 0.524\text{ ksi} < 1.2\text{ ksi} \qquad \text{allowable OK}$$

and by Eqn. 2:

$$x = \frac{A_s\, f_s}{1/2\, f_c\, b} = \frac{(6.89\text{ in}^2)(18\text{ ksi})}{1/2\,(.524\text{ ksi})(72\text{ in})} = 6.57\text{''} > 6.0 \qquad \text{assumed. Try again}$$

Try x = 6.25 in.

$$f_c = \frac{18}{10}\left(\frac{6.25}{26.61 - 6.25}\right) = 0.552 < 1.2\text{ ksi} \qquad \text{allowable OK}$$

and

$$x = \frac{(6.89)(18)}{1/2\,(.552)(72)} = 6.24 \doteq 6.25 \qquad \text{assumed OK}$$

(3) Since x = 6.24 > t = 6.0, NA is below bottom of slab and slightly into web. If web concrete in compression is neglected,

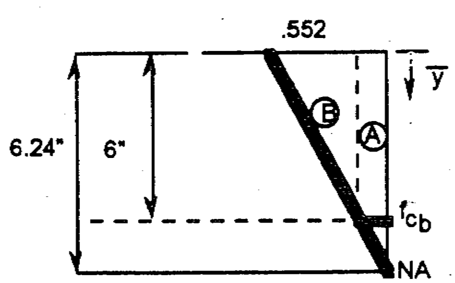
$$\text{arm} \doteq d - \frac{x}{3} \text{ for this example.}$$

$$\text{arm} \doteq 26.61 - \frac{6.24}{3} = 24.53\text{ in}$$

and capacity is

$$M = A_s\, f_s\,(\text{arm}) = (6.89)(18)(24.53) = 3042.2\text{ in-k} = 253\text{ ft-k} \qquad \text{as before.}$$

The exact "arm" may be determined from the concrete stress diagram as follows:




@ bottom of slab

$$f_{c_b} = .552\left(\frac{.24}{6.24}\right) = 0.021$$

Next find centroid of stress diagram from top of slab.

$$\bar{y} = \frac{\Sigma\, A_y}{\Sigma\, A} = \frac{(0.021)(6)(6/2) + (0.552 - 0.021)(6)(1/2)(6/3)}{(0.021)(6) + (0.552 - 0.021)(6)(1/2)}$$

$$\bar{y} = \frac{3.576}{1.722} = 2.08 \text{ in}$$

$\therefore$ arm = 26.61 - 2.08 = 24.53 in as found previously.

(4) The Operating capacity may be found as above and will be the same as for the "traditional method." The rating calculations are not shown here since they too will be the same as for the traditional method.

<u>Load Factor Rating</u> (MANUAL 6.4.2, 6.5.3 & 6.6.3)

(For this example we consider only the maximum moment section - see General Notes.)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.2.1

$$I = \frac{50}{L + 125} \le 0.30$$

$$I = \frac{50}{26 + 125} = 0.33 \text{ use } \underline{\underline{0.30}}$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1, Concrete T-Beam

$$DF = \frac{S_G}{6.0} = \frac{6.52'}{6} = 1.087$$

Thus:

$$M_{LL+I} = M_L\,(1+I) \times DF = 111.1\,(1+0.30)(1.087)$$

$$= 157 \text{ ft-k}$$

Capacity of Section - MANUAL 6.6.3.2

For unknown steel, prior to 1954 $f_y$ = 33,000 psi = 33 ksi

$M_u$ is found in accordance with applicable strength requirements of AASHTO Article 8.16.

Consider a rectangular section with compression limited to top slab. Then check MANUAL 6.6.3.2 requirement for 75% of balanced condition.

$$\rho_{max} = 0.75\,\rho_{bal} = 0.75\,\frac{0.85\,\beta_1\,f'_c}{f_y}\;\frac{87000}{87000 + f_y} \quad \text{(AASHTO Eqn. 8-18)}$$

$$\rho_{max} = 0.75\,\frac{0.85(.85)(3000)}{33000}\left(\frac{87000}{87000 + 33000}\right)$$

$$\rho_{max} = 0.0357$$

$\rho_{act} = 0.0036 << \rho_{max}$ OK (see Sheet 97)

Then:

$$a = \frac{A_s\, f_y}{0.85\, f'_c\, b_{eff}} \qquad \text{(AASHTO Eqn. 8-17)}$$

$$a = \frac{6.89\text{ in}^2\,(33\text{ ksi})}{0.85(3\text{ ksi})\; 72\text{ in}} = 1.24\text{"} < 6\text{"} \qquad \text{OK within slab}$$

$$M_R = A_s\, f_y\,(d - a/2) \qquad \text{(AASHTO Eqn. 8-16)}$$

$$M_R = (6.89\text{ in}^2)(33\text{ ksi})\left(26.61\text{ in} - \frac{1.24}{2}\right)$$

$$M_R = 5909\text{ in-k} = \underline{\underline{492\text{ ft-k}}}$$

$$M_u = \phi\, M_R \qquad \phi\text{: AASHTO 8.16.1.2.2} \rightarrow \phi = 0.90$$

$$M_u = 0.90 \times 492 = 443\text{ ft-k}$$

<u>Inventory Level</u>: MANUAL 6.5.1 & 6.6.3

$$R_I^{LF} = \frac{M_u - A_1\, M_D}{A_2 M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where in accordance with MANUAL 6.5.3

$$A_1 = 1.3$$
$$A_2 = 2.17$$

Thus:

$$RF_I^{LF} = \frac{443 - 1.3\,(109.9)}{2.17(157)} = 0.88$$

<u>Operating Level</u>: MANUAL 6.5.1 & 6.6.3

$$R_O^{LF} = \frac{M_u - A_1\, M_D}{A_2 M_{L+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where in accordance with MANUAL 6.5.3

$$\gamma_D = 1.3$$
$$\gamma_L = 1.3$$

Thus:

$$RF_O^{LF} = \frac{443 - 1.3(109.9)}{1.3(157)} = 1.47$$

Load capacity based on Load Factor Method, HS20 truck

Inventory: $0.88 \times 36^T = 31.7^T$ HS
Operating: $1.47 \times 36^T = 52.9^T$ HS

and

Inventory: $0.88 \times 1.068^x \times 20 = 18.8^T$ H (see Sheet 102)
Operating: $1.47 \times 1.068^x \times 20 = 31.4^T$ H

Load and Resistance Factor Rating (See AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*.)

(For this example we consider only the maximum moment section - see General Notes.)

Impact - Guide 3.3.2.3

Based on conditions at site (see sheet 96) select:

$$I = 0.1$$

Distribution - Guide 3.3.3 use standard AASHTO with correction factor of 1.0 (Guide Table 1).

AASHTO 3.23.2.2 and Table 3.23.1, Concrete T-Beam

$$DF = \frac{S_G}{6.0} = \frac{6.52}{6} = 1.087$$

Live Load - Guide 3.3.2.2 use HS20 to be consistent with other rating methods. Normally would use rating vehicles (Guide Fig. 2) or lane loading (Guide Fig. 3).

Thus:

$$M_{LL+I} = M_L\,(1+I)\,DF = 111.1(1+0.1)(1.087)$$
$$= 133 \text{ ft-k}$$

Capacity of Section - MANUAL 6.6.3.2

$$M_R = M_N = \frac{M_u}{\phi} \text{ found in accordance with AASHTO Article 8.16}$$

$$M_R = 492 \text{ ft-k (from sheet 105)}$$

Rating Level

$$RF^{LRF} = \frac{\phi\, M_R - A_1\, M_D}{A_2\, M_{LL+I}} \qquad \text{(MANUAL Eqn. 6-1a)}$$

where:

$\phi$ = 0.95 (Guide 3.3.4.2, Table 3(b)) From Sheet 96, concrete girder, redundant, good inspection and maintenance and good condition.

$A_1$ = 1.2 (Guide, Table 2) AC overlay measured in field (from sheet 96)

$A_2$ = 1.45 (Guide, Table 2) ADTT > 1000 and good enforcement (from sheet 96)

Then:

$$R^{LRF} = \frac{0.95(492) - 1.2(109.9)}{1.45(133)} = 1.74$$

Load Capacity Based on LRF:

$$1.74 \times 36^T = 62.6 \text{ tons HS}$$

and

$$1.74 \times 1.068 \times 20^T = 37.2^T \text{ H (see sheet 102)}$$

## SUMMARY OF RESULTS

| Method | RF | HS Truck Max. Load (tons) | H Truck Max. Load (tons) |
|---|---|---|---|
| Allowable Stress: | | | |
| Inventory | 0.91 | 32.8 | 19.4 |
| Operating | 1.54 | 55.4 | 32.9 |
| Load Factor: | | | |
| Inventory | 0.88 | 31.7 | 18.8 |
| Operating | 1.47 | 52.9 | 31.4 |
| Load and Resistance Factor | 1.74 | 62.6 | 37.2 |

Bridge No. 1895
Made By Beam
Check By ______
Date 3/4/08

MINNESOTA DEPARTMENT OF TRANSPORTATION

# ASR TIMBER ABUTMENT PILE RATING WORKSHEET

(EXAMPLE WITH NO DECAY)

**Location:** Rye Rd. over Bourbon Creek

**Given information:**
- A simple span nail laminated bridge, two lanes. Timber piles in new condition.
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built: 1970
- Timber species: Douglas fir-larch, (coastal region)

## Unit Definitions

$k \equiv 1000 \cdot lbf$ $\quad ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$ $\quad klf \equiv \frac{1000 \cdot lbf}{ft}$ $\quad kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$ $\quad kft \equiv 1000 \cdot lbf \cdot ft$ $\quad ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$ $\quad ton \equiv 2000 \cdot lbf$

**Objective :** Load rate the 12" diam. timber abutment piles driven to 20 tons

Reference Mn/DOT Spec. 3471

## Input

Reference AASHTO Table 13.5.1A

Species: Douglas fir-larch (coastal region)

Commercial Grade: Dense Select Structural

Size Class: Posts

$F_B := 1750 \cdot psi$ (bending)

$F_{Cpar} := 1150 \cdot psi$ (compression parallel to grain)

$E := 1700000 \cdot psi$ (Modulus of Elasticity)

$Dens_{timb} := 50 \cdot pcf$ (Density of timber)

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$\gamma_{EFPsoil} := 33 \cdot pcf$ (Equivalent fluid pressure of soil)

$Span := 30 \cdot ft$ (Span length, CL bearing to CL bearing)

$Width_{rdwy} := 28 \cdot ft$ (Width of roadway)

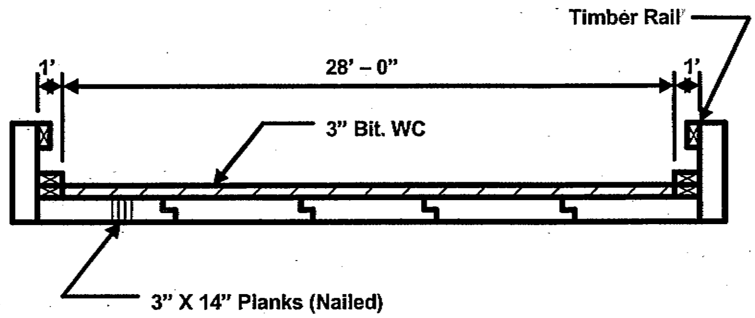
$Width_{curb} := 1 \cdot ft$ (Width of curb)

$Thick_{deck} := 14 \cdot in$ (Thickness of deck)

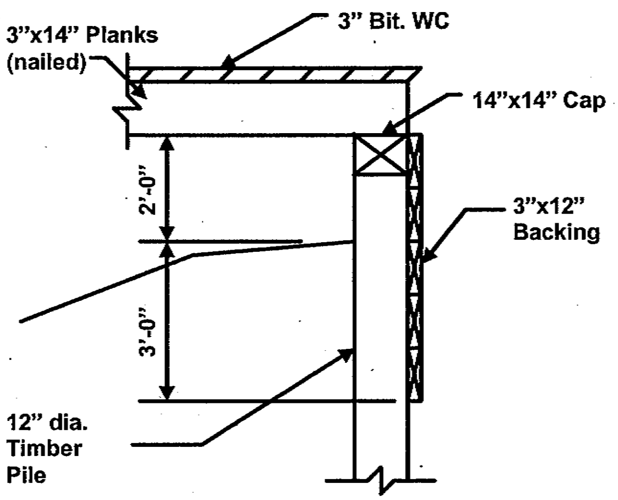
$W_{TimRail} := 50 \cdot plf$ (Weight of timber rail)

# Input (cont)

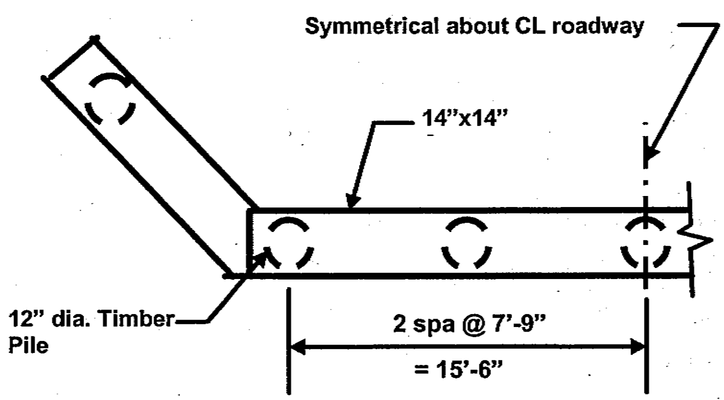
| | |
|---|---|
| $Thick_{WC} := 3\,in$ | (Thickness of Bituminous Wearing Course) |
| $PileHt_{above} := 2\cdot ft$ | (Height of pile above top of berm) |
| $PileHt_{below} := 6\cdot ft$ | (Height of pile from top of berm to assumed point of fixity) |
| $Ht_{BW} := 5\,ft$ | (Height of abutment backwall from bottom of deck downward) |
| $PileSpa := 7.75\cdot ft$ | (Pile spacing) |
| $PileDiam := 12in$ | (Pile diameter) |
| $Piles := 5$ | (Number of piles) |
| $Depth_{abutcap} := 14\cdot in$ | (Depth of abutment cap) |
| $Width_{abutcap} := 14\cdot in$ | (Width of abutment cap) |




SECTION THRU TIMBER DECK




SECTION THRU ABUTMENT




HALF PLAN OF ABUTMENT

# Dead Loads

## Superstructure

| | | |
|---|---|---|
| DL of Deck | $DL_{deck} := \left[Width_{rdwy} + \left(Width_{curb}\cdot 2\right)\right]\cdot Thick_{deck}\cdot Dens_{timb}$ | $DL_{deck} = 1750\,plf$ |
| DL of rail | $DL_{rail} := W_{TimRail}\cdot 2$ | $DL_{rail} = 100\,plf$ |
| DL of wearing course | $DL_{WC} := Width_{rdwy}\cdot Thick_{WC}\cdot Dens_{bit}$ | $DL_{WC} = 1050\,plf$ |
| **Total Dead Load of superstructure** | $DL_T := DL_{deck} + DL_{rail} + DL_{WC}$ | $DL_T = 2900\,plf$ |

## Substructure

DL of Abut. Cap

$$DL_{Abutcap} := Depth_{abutcap}\cdot Width_{abutcap}\cdot\left[\left[PileSpa\cdot(Piles - 1)\right] + (2\cdot 1.5\cdot ft)\right]\cdot Dens_{timb}$$

$$DL_{Abutcap} = 2.31\,k$$

## Compute total dead load reaction to piles

$$R_{DLTotal} := \frac{DL_T \cdot Span}{2} + DL_{Abutcap}$$

$R_{DLTotal} = 45.814\,k$

### Dead load reaction per pile

$$R_{DLpile} := \frac{R_{DLTotal}}{Piles}$$

$R_{DLpile} = 9.163\,k$

## Live Loads

### Determine number of lanes

$$Lanes := floor\left(\frac{Width_{rdwy}}{12 \cdot ft}\right)$$

$Lanes = 2$

### Reference AASHTO Appendix A for HS20-44 truck loading

$$R_{LL} := if\left[Span \le 14 \cdot ft, 32 \cdot k, if\left[Span \le 28 \cdot ft, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span}, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span} + \frac{8 \cdot k \cdot (Span - 28 \cdot ft)}{Span}\right]\right]$$

$R_{LL} = 49.6\,k$ **(Live Load End Reaction per lane due to HS20 Truck Loading)**

$R_{LLTotal} := R_{LL} \cdot Lanes$ **(Total Live Load End Reaction per abutment due to HS20 Truck Loading)** $R_{LLTotal} = 99.2\,k$

### Note: Per AASHTO 3.8.1.2 - No impact for timber members

### Live load reaction per pile

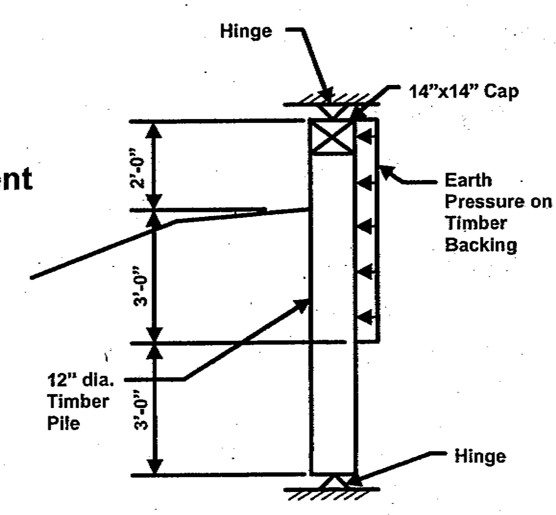
$$R_{LLpile} := \frac{R_{LLTotal}}{Piles}$$

$R_{LLpile} = 19.84\,k$

## Lateral Earth Pressure

### Assume:

- **Free draining granular fill behind backwall**
- **Equivalent fluid pressure γsoil=33 pcf**
- **Pile support point approximately 6 foot below abutment berm**
- **Hinge support conditions**




SECTION THRU PILE

$P_{Top} := \gamma_{FPsoil} \cdot (Thick_{deck} + Thick_{WC})$ **(Lateral pressure at bearing)** $P_{Top} = 46.75\,psf$

$P_{Bottom} := \gamma_{FPsoil} \cdot (PileHt_{above} + PileHt_{below})$ **(Lateral pressure at pile support point)** $P_{Bottom} = 264\,psf$

$P_{Ave} := \dfrac{P_{Top} + P_{Bottom}}{2}$ **(Average lateral pressure of $P_{Top}$ and $P_{Bottom}$)** $P_{Ave} = 155.38\,psf$

$P_{Pile} := P_{Ave} \cdot PileSpa$ **(Lateral pressure on pile)** $P_{Pile} = 1204.16\,plf$

$R_{Top} := \dfrac{P_{Pile} \cdot Ht_{BW}}{2 \cdot (PileHt_{above} + PileHt_{below})} \cdot \left[2 \cdot (PileHt_{above} + PileHt_{below}) - Ht_{BW}\right]$ **(Reaction at top of pile)** $R_{Top} = 4.14\,k$

$M_{EPress} := \dfrac{R_{Top}^{\,2}}{2 \cdot P_{Pile}}$ **(Moment in pile from earth pressure (k-ft) Ref. AISC ASD 9th Ed. Beam Diagrams No. 5, pg 2-297 )** $M_{EPress} = 7.114\,kft$

## Pile Section Properties

**Area :** $A_{pile} := \dfrac{\pi \cdot PileDiam^2}{4}$ $A_{pile} = 113.1\,in^2$

**Moment of Inertia :** $I_{pile} := \dfrac{\pi \cdot PileDiam^4}{64}$ $I_{pile} = 1017.88\,in^4$

**Section Modulus :** $S_{pile} := \dfrac{\pi \cdot PileDiam^3}{32}$ $S_{pile} = 169.65\,in^3$

**Radius of Gyration :** $r_{pile} := \dfrac{PileDiam}{4}$ $r_{pile} = 3\,in$

**Equivalent Square column dimension :** $d_{equiv} := \sqrt{A_{pile}}$ $d_{equiv} = 10.63\,in$

**Note: The load on a round pile may be taken as the same as that for a square column with the same cross sectional area.**

## Timber Adjustment Factors

**$C_M$ = Wet service factor - Art. 13.5.5.1** **(Assume all bridge timbers to exceed 19% moisture)**

$C_{Mc} := 0.91$ **(For Compression)**

$C_{Mb} := 1.0$ **(For Bending)**

**$C_D$ = Load duration factor - Art 13.5.5.2**

$C_D := 0.9$

**$C_F$ = Bending Size factor - Art. 13.5.1A Footnotes**

$C_F := 1.0$

**$C_f$ = Bending form factor for members with circular cross section- Art. 13.6.4.5**

$C_f := 1.18$

## Calculate Column Stability factor ($C_P$) - Art. 13.7.3.3

$F_{Ctab} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F$ **(Tabulated stress in compression parallel to grain)** $F_{Ctab} = 941.85\,psi$

$L_{pile} := PileHt_{above} + PileHt_{below}$ **($L_{pile}$= Actual column length between points of lateral support in inches)**

$L_{pile} = 96\,in$

$K_{pile} := 2.0$ **($K_{pile}$= Effective length factor from AASHTO Table C-1, Appendix C**

$L_{eff} := K_{pile} \cdot L_{pile}$ $L_{eff} = 192\,in$

$K_{CE} := 0.30$ **(Visually graded lumber)**

$c_{pile} := 0.85$ **(For round piles )**

$C_{ME} := 1.0$ **(Wet service factor for Mod. of Elasticity (E) - from Table 13.5.1A, for timber 5" x 5" and larger)**

$E_{Prime} := E \cdot C_{ME}$ $E_{Prime} = 1700000\,psi$

$$F_{CE} := \frac{K_{CE} \cdot (E_{Prime})}{\left(\frac{L_{eff}}{d_{equiv}}\right)^2}$$ $F_{CE} = 1564.66\,psi$

$$C_P := \left(\frac{1 + \frac{F_{CE}}{F_{Ctab}}}{2 \cdot c_{pile}}\right) - \sqrt{\frac{\left(1 + \frac{F_{CE}}{F_{Ctab}}\right)^2}{(2 \cdot c_{pile})^2} - \frac{\frac{F_{CE}}{F_{Ctab}}}{c_{pile}}}$$ **(Column Stability factor ($C_P$))** $C_P = 0.861$

**Notes:** - For short columns, crushing of wood fibers would likely control
- For Intermediate columns, crushing of wood fibers or lateral buckling may control
- For long columns, lateral buckling would likely control

## Determine allowable unit stresses for compression parallel to grain

$$F_C' = F_C \times C_M \times C_D \times C_F \times C_P$$

$F_{Cprime} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F \cdot C_P$ $F_{Cprime} = 810.96\,psi$

## Determine allowable unit stresses for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_f$$

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F \cdot C_f$ $F_{Bprime} = 1858.5\,psi$

## Determine actual bending stress

$$f_b := \frac{M_{EPress}}{S_{pile}}$$ $f_b = 503.24\,psi$

## Columns must satisfy the following for combined bending and axial compression stresses

$$f_c/F_C' + f_b/F_b' <= 1.0$$

**Solve for $f_c$ to find maximum allowed compression parallel to grain**

$$f_{cmax} := \left(1.0 - \frac{f_b}{F_{Bprime}}\right) \cdot F_{Cprime}$$ $f_{cmax} = 591.37\,\text{psi}$

## Calculate Inventory Level Rating

$P_{RI} := f_{cmax} \cdot A_{pile}$ $P_{RI} = 33.44\,\text{ton}$

$P_{RIcont} := \text{if}(P_{RI} > 20 \cdot \text{ton}, 20 \cdot \text{ton}, P_{RI})$ **(20 ton max. allowed for driving)** $P_{RIcont} = 20\,\text{ton}$ **or** $P_{RIcont} = 40\,\text{k}$

$$RF_I := \frac{P_{RIcont} - R_{DLpile}}{R_{LLpile}}$$ $RF_I = 1.55$

## Calculate Operating Level Rating

$P_{RO} := P_{RIcont} \cdot 1.33$ $P_{RO} = 53.2\,\text{k}$

$$RF_O := \frac{P_{RO} - R_{DLpile}}{R_{LLpile}}$$ $RF_O = 2.22$

### Inventory Rating

$Inv_{Rating} := RF_I \cdot 20$ **HS** $Inv_{Rating} = 31$

### Operating Rating

$Op_{Rating} := RF_O \cdot 20$ **HS** $Op_{Rating} = 44$

MINNESOTA DEPARTMENT OF TRANSPORTATION

# ASR TIMBER ABUTMENT PILE RATING WORKSHEET

(EXAMPLE WITH PILE DECAY)

**Location:** Rye Rd. over Bourbon Creek

**Given information:**
- A simple span nail laminated bridge, two lanes.
- Timber dimensions were field measured (actual)
- Good maintenance and inspection
- Year built: 1970
- Timber species: Douglas fir-larch, (coastal region)
- Piles are found to have internal decay with pile section loss at 50%.

## Unit Definitions

$k \equiv 1000 \cdot lbf$ $\quad ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$ $\quad klf \equiv \frac{1000 \cdot lbf}{ft}$ $\quad kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$ $\quad kft \equiv 1000 \cdot lbf \cdot ft$ $\quad ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$ $\quad ton \equiv 2000 \cdot lbf$

**Objective :** Load rate the 12" diam. timber abutment piles driven to 20 tons
Reference Mn/DOT Spec. 3471

## ıput

Reference AASHTO Table 13.5.1A
Species: Douglas fir-larch (coastal region)
Commercial Grade: Dense Select Structural
Size Class: Posts

$F_B := 1750 \cdot psi$ (bending)

$F_{Cpar} := 1150 \cdot psi$ (compression parallel to grain)

$E := 1700000 \cdot psi$ (Modulus of Elasticity)

$Dens_{timb} := 50 \cdot pcf$ (Density of timber)

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$\gamma_{FPsoil} := 33 \cdot pcf$ (Equivalent fluid pressure of soil)

$Span := 30 \cdot ft$ (Span length, CL bearing to CL bearing)

$Width_{rdwy} := 28 \cdot ft$ (Width of roadway)

$Width_{curb} := 1 \cdot ft$ (Width of curb)

$Thick_{deck} := 14 \cdot in$ (Thickness of deck)

$W_{TimRail} := 50 \cdot plf$ (Weight of timber rail)

# Input (cont)



$Thick_{WC} := 3 \cdot in$ (Thickness of Bituminous Wearing Course)

$PileHt_{above} := 2 \cdot ft$ (Height of pile above top of berm)

$PileHt_{below} := 6 \cdot ft$ (Height of pile from top of berm to assumed point of fixity)

$Ht_{BW} := 5 \cdot ft$ (Height of abutment backwall from bottom of deck downward)

$PileSpa := 7.75 \cdot ft$ (Pile spacing)

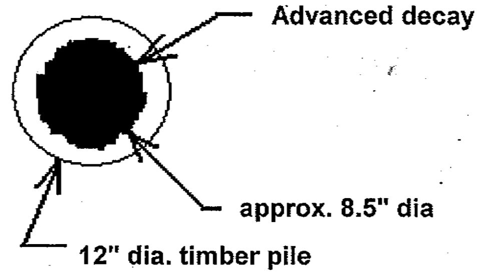
$PileDiam := 12 in$ (Pile diameter)

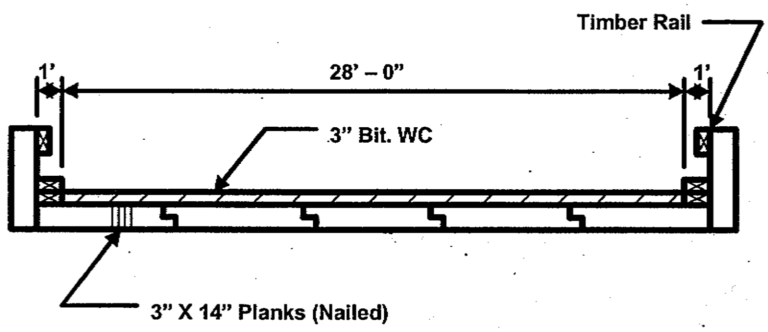
$PileDiam_{decay} := 8.5 \cdot in$ (Diameter of internal decayed section)

$Piles := 5$ (Number of piles)

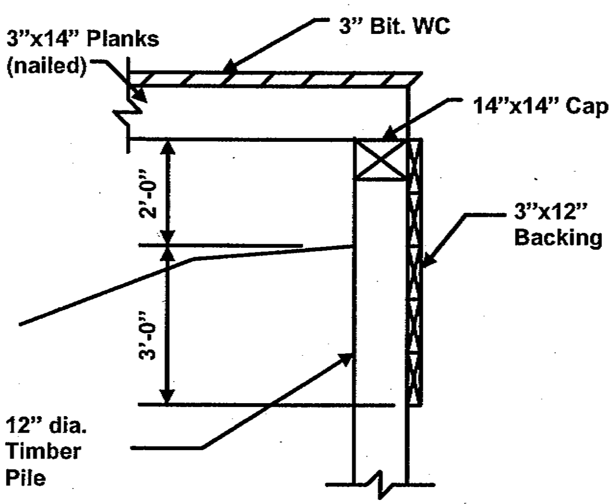
$Depth_{abutcap} := 14 \cdot in$ (Depth of abutment cap)

$Width_{abutcap} := 14 \cdot in$ (Width of abutment cap)

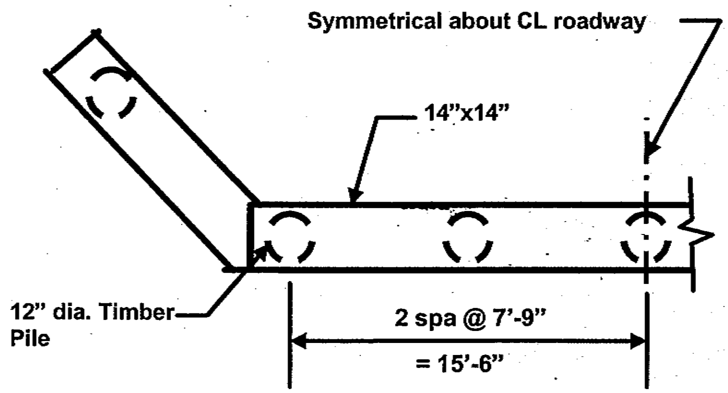



SECTION THRU TIMBER DECK




SECTION THRU ABUTMENT




HALF PLAN OF ABUTMENT

## Dead Loads

### Superstructure

| | | |
|---|---|---|
| **DL of Deck** | $DL_{deck} := \left[Width_{rdwy} + \left(Width_{curb} \cdot 2\right)\right] \cdot Thick_{deck} \cdot Dens_{timb}$ | $DL_{deck} = 1750\,plf$ |
| **DL of rail** | $DL_{rail} := W_{TimRail} \cdot 2$ | $DL_{rail} = 100\,plf$ |
| **DL of wearing course** | $DL_{WC} := Width_{rdwy} \cdot Thick_{WC} \cdot Dens_{bit}$ | $DL_{WC} = 1050\,plf$ |
| **Total Dead Load of superstructure** | $DL_T := DL_{deck} + DL_{rail} + DL_{WC}$ | $DL_T = 2900\,plf$ |

## Substructure

**DL of Abut. Cap** $DL_{Abutcap} := Depth_{abutcap} \cdot Width_{abutcap} \cdot [[PileSpa \cdot (Piles - 1)] + (2 \cdot 1.5 \cdot ft)] \cdot Dens_{timb}$

$DL_{Abutcap} = 2.31\,k$

## Compute total dead load reaction to piles

$$R_{DLTotal} := \frac{DL_T \cdot Span}{2} + DL_{Abutcap} \qquad R_{DLTotal} = 45.81\,k$$

## Dead load reaction per pile

$$R_{DLpile} := \frac{R_{DLTotal}}{Piles} \qquad R_{DLpile} = 9.16\,k$$

## Live Loads

### Determine number of lanes

$$Lanes := floor\left(\frac{Width_{rdwy}}{12 \cdot ft}\right) \qquad Lanes = 2$$

### Reference AASHTO Appendix A for HS20-44 truck loading

$$R_{LL} := if\left[Span \le 14 \cdot ft, 32 \cdot k, if\left[Span \le 28 \cdot ft, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span}, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span} + \frac{8 \cdot k \cdot (Span - 28 \cdot ft)}{Span}\right]\right]$$

$R_{LL} = 49.6\,k$ **(Live Load End Reaction per lane due to HS20 Truck Loading)**

$R_{LLTotal} := R_{LL} \cdot Lanes$ **(Total Live Load End Reaction per abutment due to HS20 Truck Loading)** $R_{LLTotal} = 99.2\,k$

### Note: Per AASHTO 3.8.1.2 - No impact for timber members

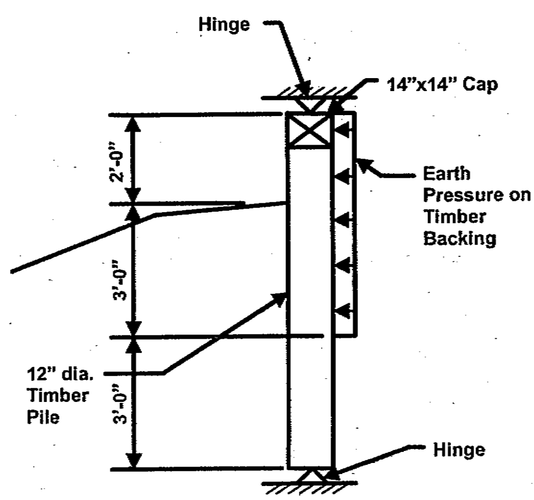
## Live load reaction per pile

$$R_{LLpile} := \frac{R_{LLTotal}}{Piles} \qquad R_{LLpile} = 19.84\,k$$

## Lateral Earth Pressure

### Assume:

- Free draining granular fill behind backwall
- Equivalent fluid pressure γsoil=33 pcf
- Pile support point approximately 6 foot below abutment berm
- Hinge support conditions




SECTION THRU PILE

$P_{Top} := \gamma_{FPsoil} \cdot (Thick_{deck} + Thick_{WC})$ **(Lateral pressure at bearing)** $P_{Top} = 46.75\,psf$

$P_{Bottom} := \gamma_{FPsoil} \cdot (PileHt_{above} + PileHt_{below})$ **(Lateral pressure at pile support point)** $P_{Bottom} = 264\,psf$

$P_{Ave} := \dfrac{P_{Top} + P_{Bottom}}{2}$ **(Average lateral pressure of $P_{Top}$ and $P_{Bottom}$)** $P_{Ave} = 155.38\,psf$

$P_{Pile} := P_{Ave} \cdot PileSpa$ **(Lateral pressure on pile)** $P_{Pile} = 1204.16\,plf$

$R_{Top} := \dfrac{P_{Pile} \cdot Ht_{BW}}{2 \cdot (PileHt_{above} + PileHt_{below})} \cdot \left[2 \cdot (PileHt_{above} + PileHt_{below}) - Ht_{BW}\right]$ **(Reaction at top of pile)** $R_{Top} = 4.14\,k$

$M_{EPress} := \dfrac{R_{Top}^{\;2}}{2 \cdot P_{Pile}}$ **(Moment in pile from earth pressure (k-ft) Ref. AISC ASD 9th Ed. Beam Diagrams No. 5, pg 2-297 )** $M_{EPress} = 7.114\,kft$

## Pile Section Properties

**Area :** $A_{pile} := \dfrac{\pi \cdot \left(PileDiam^2 - PileDiam_{decay}^{\;2}\right)}{4}$ $A_{pile} = 56.35\,in^2$

**Moment of Inertia :** $I_{pile} := \dfrac{\pi \cdot \left(PileDiam^4 - PileDiam_{decay}^{\;4}\right)}{64}$ $I_{pile} = 761.64\,in^4$

**Section Modulus :** $S_{pile} := \dfrac{\pi \cdot \left(PileDiam^4 - PileDiam_{decay}^{\;4}\right)}{32 \cdot PileDiam}$ $S_{pile} = 126.94\,in^3$

**Radius of Gyration :** $r_{pile} := \dfrac{\sqrt{PileDiam^2 + PileDiam_{decay}^{\;2}}}{4}$ $r_{pile} = 3.676\,in$

**Equivalent Square column dimension :** $d_{equiv} := \sqrt{A_{pile}}$ $d_{equiv} = 7.51\,in$

**Note: The load on a round pile may be taken as the same as that for a square column with the same cross sectional area.**

## Timber Adjustment Factors

**$C_M$ = Wet service factor - Art. 13.5.5.1** **(Assume all bridge timbers to exceed 19% moisture)**

$C_{Mc} := 0.91$ **(For Compression)**

$C_{Mb} := 1.0$ **(For Bending)**

**$C_D$ = Load duration factor - Art 13.5.5.2**

$C_D := 0.9$

**$C_F$ = Bending Size factor - Art. 13.5.1A Footnotes**

$C_F := 1.0$

**$C_f$ = Bending form factor for members with circular cross section- Art. 13.6.4.5**

$C_f := 1.18$

## Calculate Column Stability factor ($C_P$) - Art. 13.7.3.3

$F_{Ctab} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F$ **(Tabulated stress in compression parallel to grain)** $F_{Ctab} = 941.85 \text{ psi}$

$L_{pile} := PileHt_{above} + PileHt_{below}$ **($L_{pile}$= Actual column length between points of lateral support in inches)**

$L_{pile} = 96 \text{ in}$

$K_{pile} := 2.0$ **($K_{pile}$= Effective length factor from AASHTO Table C-1, Appendix C**

$L_{eff} := K_{pile} \cdot L_{pile}$ $L_{eff} = 192 \text{ in}$

$K_{CE} := 0.30$ **(Visually graded lumber)**

$c_{pile} := 0.85$ **(For round piles )**

$C_{ME} := 1.0$ **(Wet service factor for Mod. of Elasticity (E) - from Table 13.5.1A, for timber 5" x 5" and larger)**

$E_{Prime} := E \cdot C_{ME}$ $E_{Prime} = 1700000 \text{ psi}$

$$F_{CE} := \frac{K_{CE} \cdot (E_{Prime})}{\left(\frac{L_{eff}}{d_{equiv}}\right)^2} \qquad F_{CE} = 779.61 \text{ psi}$$

$$C_P := \left(\frac{1 + \frac{F_{CE}}{F_{Ctab}}}{2 \cdot c_{pile}}\right) - \sqrt{\frac{\left(1 + \frac{F_{CE}}{F_{Ctab}}\right)^2}{(2 \cdot c_{pile})^2} - \frac{\frac{F_{CE}}{F_{Ctab}}}{c_{pile}}}$$

**(Column Stability factor ($C_P$))** $C_P = 0.648$

**Notes:** - For short columns, crushing of wood fibers would likely control
- For Intermediate columns, crushing of wood fibers or lateral buckling may control
- For long columns, lateral buckling would likely control

## Determine allowable unit stresses for compression parallel to grain

$$F_C' = F_C \times C_M \times C_D \times C_F \times C_P$$

$F_{Cprime} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F \cdot C_P$ $F_{Cprime} = 610.69 \text{ psi}$

## Determine allowable unit stresses for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_f$$

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F$ **Note: $C_f$ was set to 1.0 conservatively due to rotting** $F_{Bprime} = 1575 \text{ psi}$

## Determine actual bending stress

$$f_b := \frac{M_{EPress}}{S_{pile}} \qquad f_b = 672.55 \text{ psi}$$

## Columns must satisfy the following for combined bending and axial compression stresses

$$f_c/F_C' + f_b/F_b' <= 1.0$$

**Solve for $f_c$ to find maximum allowed compression parallel to grain**

$$f_{cmax} := \left(1.0 - \frac{f_b}{F_{Bprime}}\right) \cdot F_{Cprime}$$ $f_{cmax} = 349.92\ \text{psi}$

## Calculate Inventory Level Rating

$P_{RI} := f_{cmax} \cdot A_{pile}$ $P_{RI} = 9.86\ \text{ton}$

$P_{RIcont} := \text{if}(P_{RI} > 20 \cdot \text{ton}, 20 \cdot \text{ton}, P_{RI})$ **(20 ton max. allowed for driving)** $P_{RIcont} = 9.86\ \text{ton}$ **or** $P_{RIcont} = 19.719\ \text{k}$

$$RF_I := \frac{P_{RIcont} - R_{DLpile}}{R_{LLpile}}$$ $RF_I = 0.53$

## Calculate Operating Level Rating

$P_{RO} := P_{RIcont} \cdot 1.33$ $P_{RO} = 26.23\ \text{k}$

$$RF_O := \frac{P_{RO} - R_{DLpile}}{R_{LLpile}}$$ $RF_O = 0.86$

### Inventory Rating

$Inv_{Rating} := RF_I \cdot 20$ **HS** $Inv_{Rating} = 11$

### Operating Rating

$Op_{Rating} := RF_O \cdot 20$ **HS** $Op_{Rating} = 17$

MINNESOTA DEPARTMENT OF TRANSPORTATION

# ASR TIMBER ABUTMENT PILE AND PILE CAP RATING WORKSHEET

(EXAMPLE WITH LOSS OF PILE)

Sheet No. 1 of 11
Bridge No. 1895
Made By Beam
Check By ______
Date 3/4/08

---

**Location:** Rye Rd. over Bourbon Creek

**Given information:**
- A simple span nail laminated bridge, two lanes.
- Timber dimensions were field measured (actual)
- Poor maintenance and inspection
- Year built: 1970
- Timber species: Douglas fir-larch, (coastal region)
- First interior piles (pile "B") are found to have substantial internal decay with pile section loss at 100%.

## Unit Definitions

$k \equiv 1000 \cdot lbf$   $ksf \equiv 1000 \cdot \frac{lbf}{ft^2}$   $klf \equiv \frac{1000 \cdot lbf}{ft}$   $kcf \equiv 1000 \cdot \frac{lbf}{ft^3}$   $kft \equiv 1000 \cdot lbf \cdot ft$   $ksi \equiv \frac{1000 \cdot lbf}{in \cdot in}$   $ton \equiv 2000 \cdot lbf$

## Input

**Piles** From AASHTO Table 13.5.1A

Species: Douglas fir-larch (coastal region)
Commercial Grade: Dense Select Structural
Size Class: Posts

$F_B := 1750 \cdot psi$ (bending)

$F_{Cpar} := 1150 \cdot psi$ (compression parallel to grain)

$E := 1700000 \cdot psi$ (Modulus of Elasticity)

$Dens_{timb} := 50 \cdot pcf$ (Density of timber)

$Dens_{bit} := 150 \cdot pcf$ (Density of bituminous)

$\gamma_{EPsoil} := 33 \cdot pcf$ (Equivalent fluid pressure of soil)

**Abutment Cap** From AASHTO Table 13.5.1A

Species: Douglas fir-larch (coastal region)
Commercial Grade: Select Structural
Size Class: Beams and stringers

$F_{Bcap} := 1600 \cdot psi$ (bending)

$F_{Vpar} := 85 \cdot psi$ (shear parallel to grain)

**Objective :** Load rate the 12" diam. timber abutment piles driven to 20 tons

Reference Mn/DOT Spec. 3471

## Input (cont)

$Span := 30\ ft$ (Span length, CL bearing to CL bearing)

$Width_{rdwy} := 28\ ft$ (Width of roadway)

$Width_{curb} := 1\ ft$ (Width of curb)

$Thick_{deck} := 14\ in$ (Thickness of deck)

$W_{TimRail} := 50\ plf$ (Weight of timber rail)

$Thick_{WC} := 3\ in$ (Thickness of Bituminous Wearing Course)

$PileHt_{above} := 2\ ft$ (Height of pile above top of berm)

$PileHt_{below} := 6\ ft$ (Height of pile from top of berm to assumed point of fixity)

$Ht_{BW} := 5\ ft$ (Height of abutment backwall from bottom of deck downward)

$PileSpa := 7.75\ ft$ (Pile spacing)

$PileDiam := 12 in$ (Pile diameter)

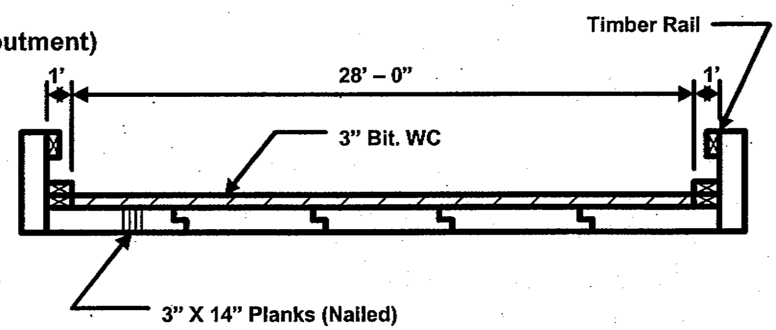
$PileDiam_{decay} := 0\ in$ (Diameter of internal decayed section)

$Piles := 5$ (Total Number of piles per abutment)

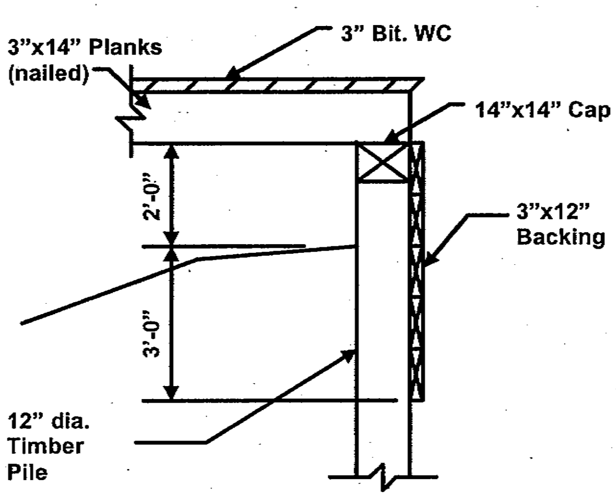
$Piles_{rotted} := 2$ (Number of rotted piles at abutment)

$Depth_{abutcap} := 14\ in$ (Depth of abutment cap)

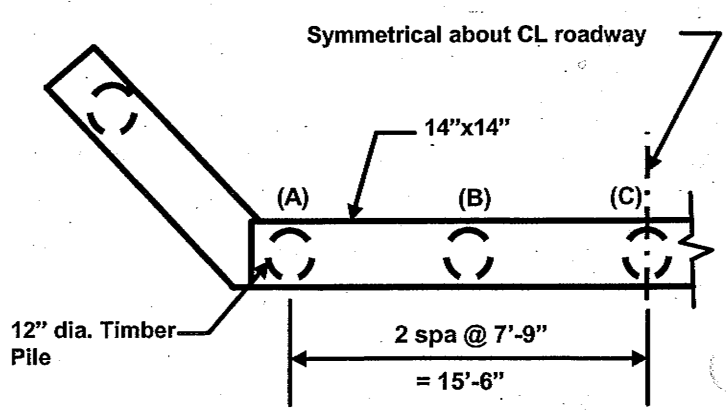
$Width_{abutcap} := 14\ in$ (Width of abutment cap)




SECTION THRU TIMBER DECK




SECTION THRU ABUTMENT




HALF PLAN OF ABUTMENT

# Dead Loads

## Superstructure

**DL of Deck** $DL_{deck} := \left[Width_{rdwy} + \left(Width_{curb} \cdot 2\right)\right] \cdot Thick_{deck} \cdot Dens_{timb}$ $DL_{deck} = 1750\,plf$

**DL of rail** $DL_{rail} := W_{TimRail} \cdot 2$ $DL_{rail} = 100\,plf$

**DL of wearing course** $DL_{WC} := Width_{rdwy} \cdot Thick_{WC} \cdot Dens_{bit}$ $DL_{WC} = 1050\,plf$

**Total Dead Load of superstructure** $DL_T := DL_{deck} + DL_{rail} + DL_{WC}$ $DL_T = 2900\,plf$

## Substructure

**DL of Abut. Cap** $DL_{Abutcap} := Depth_{abutcap} \cdot Width_{abutcap} \cdot [[PileSpa \cdot (Piles - 1)] + (2 \cdot 1.5 \cdot ft)] \cdot Dens_{timb}$

$DL_{Abutcap} = 2.31\,k$

## Compute total dead load reaction to piles

$$R_{DLTotal} := \frac{DL_T \cdot Span}{2} + DL_{Abutcap}$$ $R_{DLTotal} = 45.81\,k$

## Dead load reaction per pile

$$R_{DLpile} := \frac{R_{DLTotal}}{Piles - Piles_{rotted}}$$ $R_{DLpile} = 15.27\,k$

# Live Loads

## Determine number of lanes

$$Lanes := floor\left(\frac{Width_{rdwy}}{12 \cdot ft}\right)$$ $Lanes = 2$

## Reference AASHTO Appendix A for HS20-44 truck loading

$$R_{LL} := if\left[Span \le 14 \cdot ft, 32 \cdot k, if\left[Span \le 28 \cdot ft, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span}, 32 \cdot k + \frac{32 \cdot k \cdot (Span - 14 \cdot ft)}{Span} + \frac{8 \cdot k \cdot (Span - 28 \cdot ft)}{Span}\right]\right]$$

$R_{LL} = 49.6\,k$ **(Live Load End Reaction per lane due to HS20 Truck Loading)**

$R_{LLTotal} := R_{LL} \cdot Lanes$ **(Total Live Load End Reaction per abutment due to HS20 Truck Loading)** $R_{LLTotal} = 99.2\,k$

## Note: Per AASHTO 3.8.1.2 - No impact for timber members

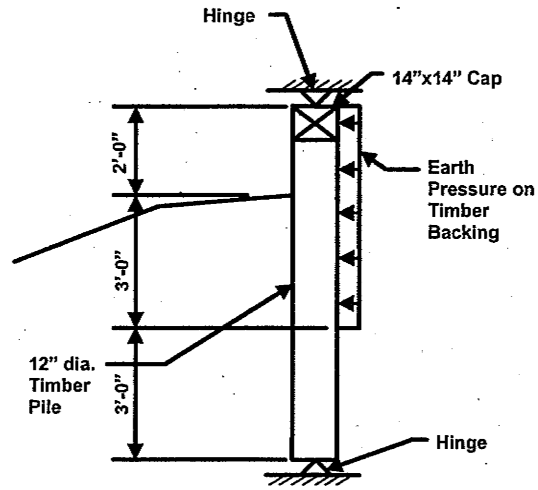
## Live load reaction per pile

$$R_{LLpile} := \frac{R_{LLTotal}}{Piles - Piles_{rotted}}$$ $R_{LLpile} = 33.067\,k$

## Lateral Earth Pressure

### Assume

- **Free draining granular fill behind backwall**
- **Equivalent fluid pressure γsoil=33 pcf**
- **Pile support point approximately 6 foot below abutment berm**
- **Hinge support conditions**




SECTION THRU PILE

$P_{Top} := \gamma_{FPsoil} \cdot (Thick_{deck} + Thick_{WC})$ **(Lateral pressure at bearing)** $P_{Top} = 46.75\,psf$

$P_{Bottom} := \gamma_{FPsoil} \cdot (PileHt_{above} + PileHt_{below})$ **(Lateral pressure at pile support point)** $P_{Bottom} = 264\,psf$

$P_{Ave} := \dfrac{P_{Top} + P_{Bottom}}{2}$ **(Average lateral pressure of $P_{Top}$ and $P_{Bottom}$)** $P_{Ave} = 155.38\,psf$

$P_{Pile} := P_{Ave} \cdot PileSpa$ **(Lateral pressure on pile)** $P_{Pile} = 1204.16\,plf$

$R_{Top} := \dfrac{P_{Pile} \cdot Ht_{BW}}{2 \cdot (PileHt_{above} + PileHt_{below})} \cdot \left[2 \cdot (PileHt_{above} + PileHt_{below}) - Ht_{BW}\right]$ **(Reaction at top of pile)** $R_{Top} = 4.14\,k$

$M_{EPress} := \dfrac{R_{Top}^{\,2}}{2 \cdot P_{Pile}}$ **(Moment in pile from earth pressure (k-ft) Ref. AISC ASD 9th Ed. Beam Diagrams No. 5, pg 2-297 )** $M_{EPress} = 7.114\,kft$

## Pile Section Properties

**Area :** $A_{pile} := \dfrac{\pi \cdot \left(PileDiam^2 - PileDiam_{decay}^{\,2}\right)}{4}$ $A_{pile} = 113.1\,in^2$

**Moment of Inertia :** $I_{pile} := \dfrac{\pi \cdot \left(PileDiam^4 - PileDiam_{decay}^{\,4}\right)}{64}$ $I_{pile} = 1017.88\,in^4$

**Section Modulus :** $S_{pile} := \dfrac{\pi \cdot \left(PileDiam^4 - PileDiam_{decay}^{\,4}\right)}{32 \cdot PileDiam}$ $S_{pile} = 169.65\,in^3$

**Radius of Gyration :** $r_{pile} := \dfrac{\sqrt{PileDiam^2 + PileDiam_{decay}^{\,2}}}{4}$ $r_{pile} = 3\,in$

**Equivalent Square column dimension :** $d_{equiv} := \sqrt{A_{pile}}$ $d_{equiv} = 10.63\,\text{in}$

**Note: The load on a round pile may be taken as the same as that for a square column with the same cross sectional area.**

## Timber Adjustment Factors

**$C_M$ = Wet service factor - Art. 13.5.5.1 & Table 13.5.1A (Assume all bridge timbers to exceed 19% moisture)**

$C_{Mc} := 0.91$ **(For Compression)**

$C_{Mb} := 1.0$ **(For Bending)**

**$C_D$ = Load duration factor - Art 13.5.5.2**

$C_D := 0.9$

**$C_F$ = Bending Size factor - Art. 13.5.1A Footnotes**

$C_F := 1.0$

**$C_f$ = Bending form factor for members with circular cross section- Art. 13.6.4.5**

$C_f := 1.18$

## Calculate Column Stability factor ($C_P$) - Art. 13.7.3.3

$F_{Ctab} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F$ **(Tabulated stress in compression parallel to grain)** $F_{Ctab} = 941.85\,\text{psi}$

$L_{pile} := PileHt_{above} + PileHt_{below}$ **($L_{pile}$= Actual column length between points of lateral support in inches)**

$L_{pile} = 96\,\text{in}$

$K_{pile} := 2.0$ **($K_{pile}$= Effective length factor from AASHTO Table C-1, Appendix C**

$L_{eff} := K_{pile} \cdot L_{pile}$ $L_{eff} = 192\,\text{in}$

$K_{CE} := 0.30$ **(Visually graded lumber)**

$c_{pile} := 0.85$ **(For round piles )**

$C_{ME} := 1.0$ **(Wet service factor for Mod. of Elasticity (E) - from Table 13.5.1A, for timber 5" x 5" and larger)**

$E_{Prime} := E \cdot C_{ME}$ $E_{Prime} = 1700000\,\text{psi}$

$$F_{CE} := \frac{K_{CE} \cdot (E_{Prime})}{\left(\frac{L_{eff}}{d_{equiv}}\right)^2}$$ $F_{CE} = 1564.66\,\text{psi}$

$$C_P := \left(\frac{1 + \frac{F_{CE}}{F_{Ctab}}}{2 \cdot c_{pile}}\right) - \sqrt{\frac{\left(1 + \frac{F_{CE}}{F_{Ctab}}\right)^2}{(2 \cdot c_{pile})^2} - \frac{\frac{F_{CE}}{F_{Ctab}}}{c_{pile}}}$$ **(Column Stability factor ($C_P$))** $C_P = 0.861$

**Notes:** **- For short columns, crushing of wood fibers would likely control**
**- For Intermediate columns, crushing of wood fibers or lateral buckling may control**
**- For long columns, lateral buckling would likely control**

## Determine allowable unit stresses for compression parallel to grain

$$F_C' = F_C \times C_M \times C_D \times C_F \times C_P$$

$F_{Cprime} := F_{Cpar} \cdot C_{Mc} \cdot C_D \cdot C_F \cdot C_P$ $\qquad F_{Cprime} = 810.96\,\text{psi}$

## Determine allowable unit stresses for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_f$$

$F_{Bprime} := F_B \cdot C_{Mb} \cdot C_D \cdot C_F$ **Note: $C_f$ was set to 1.0 conservatively due to rotting** $\qquad F_{Bprime} = 1575\,\text{psi}$

## Determine actual bending stress

$$f_b := \frac{M_{EPress}}{S_{pile}} \qquad f_b = 503.24\,\text{psi}$$

## Columns must satisfy the following for combined bending and axial compression stresses

$$f_c/F_C' + f_b/F_b' <= 1.0$$

**Solve for $f_c$ to find maximum allowed compression parallel to grain**

$$f_{cmax} := \left(1.0 - \frac{f_b}{F_{Bprime}}\right) \cdot F_{Cprime} \qquad f_{cmax} = 551.85\,\text{psi}$$

## Calculate Inventory Level Rating

$P_{RI} := f_{cmax} \cdot A_{pile}$ $\qquad P_{RI} = 31.21\,\text{ton}$

$P_{RIcont} := \text{if}\left(P_{RI} > 20 \cdot \text{ton}, 20 \cdot \text{ton}, P_{RI}\right)$ **(20 ton max. allowed for driving)** $\qquad P_{RIcont} = 20\,\text{ton}$ **or** $P_{RIcont} = 40\,\text{k}$

$$RF_I := \frac{P_{RIcont} - R_{DLpile}}{R_{LLpile}} \qquad RF_I = 0.75$$

## Calculate Operating Level Rating for piles

$P_{RO} := P_{RIcont} \cdot 1.33$ $\qquad P_{RO} = 53.2\,\text{k}$

$$RF_O := \frac{P_{RO} - R_{DLpile}}{R_{LLpile}} \qquad RF_O = 1.15$$

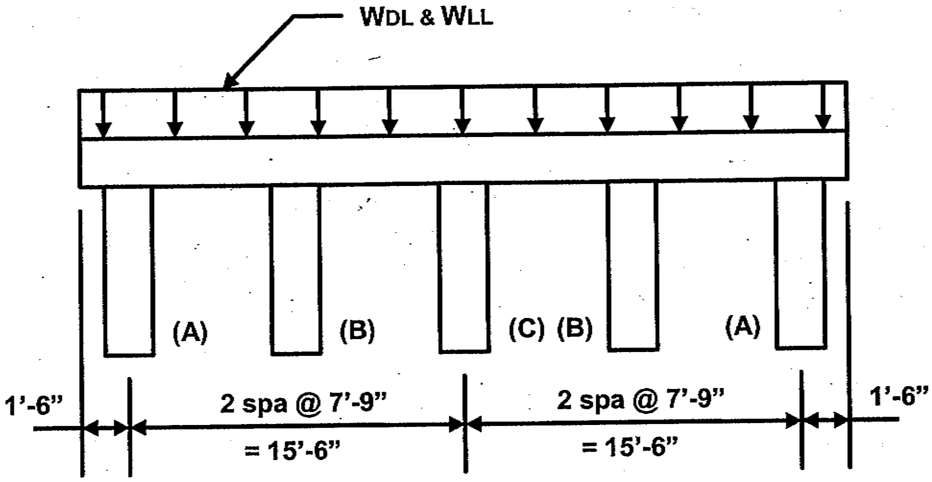
### Inventory Rating

$Inv_{Rating} := RF_I \cdot 20$ $\qquad$ **HS** $Inv_{Rating} = 15$

### Operating Rating

$Op_{Rating} := RF_O \cdot 20$ $\qquad$ **HS** $Op_{Rating} = 23$

## Check Rating based on 14" x 14" abutment cap




## Dead Load on abutment cap

$$W_{DLcap} := \frac{R_{DLTotal}}{[PileSpa \cdot (Piles - 1)] + (2 \cdot 1.5 \cdot ft)}$$ **(Dead Load per foot of cap)** $W_{DLcap} = 1.35\,klf$

## Live Load on abutment cap

$$W_{LLcap} := \frac{R_{LLTotal}}{[PileSpa \cdot (Piles - 1)] + (2 \cdot 1.5 \cdot ft)}$$ **(Live Load per foot of cap - 2 lanes of HS20 truck loading)** $W_{LLcap} = 2.92\,klf$

**- Reference continuous beam charts for two equal spans with uniform load on both spans**
**- Moment maximum at center pile support (C).**

## Moments in abutment cap

$$M_{DLSupC} := \frac{W_{DLcap} \cdot (PileSpa \cdot 2)^2}{8}$$ **(Dead load moment at pile support)** $M_{DLSupC} = 40.47\,kft$

$$M_{LLSupC} := \frac{W_{LLcap} \cdot (PileSpa \cdot 2)^2}{8}$$ **(Live load moment at pile support)** $M_{LLSupC} = 87.62\,kft$

## Shear in abutment cap

$$V_{DLSupC} := \frac{5}{8} W_{DLcap} \cdot (PileSpa \cdot 2)$$ **(Dead load shear at CL pile support)** $V_{DLSupC} = 13.05\,k$

$$V_{LLSupC} := \frac{5}{8} W_{LLcap} \cdot (PileSpa \cdot 2)$$ **(Live load shear at CL pile support)** $V_{LLSupC} = 28.26\,k$

**- Per AASHTO 13.6.5.2 - For uniformly distributed loads, the maximum shear occurs at a distance from the support equal to the bending member depth "d".**

**- Therefore, check shear at:** $x_{shear} := (PileSpa \cdot 2) - \frac{PileDiam}{2} - Depth_{abutcap}$ $\quad x_{shear} = 13.83\,ft$

$$V_{DLSupd} := V_{DLSupC} \cdot \frac{x_{shear} - \left[\frac{3}{8} \cdot (PileSpa \cdot 2)\right]}{(PileSpa \cdot 2) - \left[\frac{3}{8} \cdot (PileSpa \cdot 2)\right]}$$ **(Dead load shear at "d" from pile support)** $V_{DLSupd} = 10.81\,k$

$$V_{LLSupd} := V_{LLSupC} \cdot \frac{x_{shear} - \left[\frac{3}{8} \cdot (PileSpa \cdot 2)\right]}{(PileSpa \cdot 2) - \left[\frac{3}{8} \cdot (PileSpa \cdot 2)\right]}$$ **(Live load shear at "d" from pile support)** $V_{LLSupd} = 23.4\,k$

## Abutment Cap Section Properties

**Area :** $A_{Abutcap} := Width_{abutcap} \cdot Depth_{abutcap}$ $\quad A_{Abutcap} = 196\,in^2$

**Moment of Inertia :** $I_{Abutcap} := \frac{Width_{abutcap} \cdot Depth_{abutcap}^{\;3}}{12}$ $\quad I_{Abutcap} = 3201.33\,in^4$

**Section Modulus :** $S_{Abutcap} := \frac{I_{Abutcap}}{\frac{Depth_{abutcap}}{2}}$ $\quad S_{Abutcap} = 457.33\,in^3$

## Timber Adjustment Factors

**$C_M$ = Wet service factor - Art. 13.5.5.1 & Table 13.5.1A (Assume all bridge timbers to exceed 19% moisture)**

$C_{Mb} := 1.0$ **(For Bending - Timbers 5" x 5" or larger)**

$C_{Mv} := 1.0$ **(For Shear - Timbers 5" x 5" or larger)**

**$C_D$ = Load duration factor - Table 13.5.5A (Veh LL, 2 months)**

$C_D := 1.15$

**$C_F$ = Bending Size factor - For lumber thicker than 5" - Art. 13.6.4.2.2**

$$C_F := \left(\frac{12 \cdot in}{Depth_{abutcap}}\right)^{\frac{1}{9}} \qquad C_F = 0.98$$

**$C_L$ = Beam Stability factor from Art. 13.6.4.4**

$C_L := 1.0$ **(Assume no danger of lateral buckling)**

## Determine allowable unit stresses for bending

$$F_B' = F_B \times C_M \times C_D \times C_F \times C_L$$

$F_{BprimeCap} := F_{Bcap} \cdot C_{Mb} \cdot C_D \cdot C_F \cdot C_L$ $\quad F_{BprimeCap} = 1808.75\,psi$

## Determine allowable unit stresses for shear

$$F_V' = F_V \times C_M \times C_D$$

$F_{VprimeCap} := F_{Vpar} \cdot C_{Mv} \cdot C_D$ $\qquad F_{VprimeCap} = 97.75\,\text{psi}$

# Check Inventory and Operating Rating of Abutment Cap

## Calculate Inventory and Operating Stresses

### Bending

$F_{BInvCap} := F_{BprimeCap}$ **(Inventory stress = Allowable unit stress )** $\qquad F_{BInvCap} = 1808.75\,\text{psi}$

$F_{BOpCap} := F_{BprimeCap} \cdot 1.33$ **(Operating stress)** $\qquad F_{BOpCap} = 2405.64\,\text{psi}$

### Shear

$F_{VInvCap} := F_{VprimeCap}$ **(Inventory stress = Allowable unit stress )** $\qquad F_{VInvCap} = 97.75\,\text{psi}$

$F_{VOpCap} := F_{VprimeCap} \cdot 1.33$ **(Operating stress)** $\qquad F_{VOpCap} = 130.01\,\text{psi}$

## Calculate Inventory Rating Factor for Bending in Cap

$M_{RIb} := F_{BInvCap} \cdot S_{Abutcap}$ $\qquad M_{RIb} = 68.93\,\text{kft}$

$$RF_{Ib} := \frac{M_{RIb} - M_{DLSupC}}{M_{LLSupC}} \qquad RF_{Ib} = 0.32$$

## Calculate Operating Rating Factor for Bending in Cap

$M_{ROb} := F_{BOpCap} \cdot S_{Abutcap}$ $\qquad M_{ROb} = 91.68\,\text{kft}$

$$RF_{Ob} := \frac{M_{ROb} - M_{DLSupC}}{M_{LLSupC}} \qquad RF_{Ob} = 0.58$$

## Inventory Rating for Bending in Cap

$Inv_{RatingB} := RF_{Ib} \cdot 20$ $\qquad$ **HS** $Inv_{RatingB} = 6$

## Operating Rating for Bending in Cap

$Op_{RatingB} := RF_{Ob} \cdot 20$ $\qquad$ **HS** $Op_{RatingB} = 12$

## Calculate Inventory Rating Factor for Shear in Cap

$V_{RI} := \frac{2}{3} \cdot A_{Abutcap} \cdot F_{VInvCap}$ $\qquad V_{RI} = 12.77\,\text{k}$

$$RF_{IV} := \frac{V_{RI} - V_{DLSupd}}{V_{LLSupd}} \qquad RF_{IV} = 0.08$$

## Calculate Operating Rating Factor for Shear in Cap

$$V_{RO} := \frac{2}{3} \cdot A_{Abutcap} \cdot F_{VOpCap} \qquad V_{RO} = 16.99\,k$$

$$RF_{OV} := \frac{V_{RO} - V_{DLSupd}}{V_{LLSupd}} \qquad RF_{OV} = 0.26$$

### Inventory Rating for Shear in Cap

$$Inv_{RatingV} := RF_{IV} \cdot 20 \qquad \text{HS } Inv_{RatingV} = 2$$

### Operating Rating for Shear in Cap

$$Op_{RatingV} := RF_{OV} \cdot 20 \qquad \text{HS } Op_{RatingV} = 5$$

## For comparison purposes, check Inventory and Operating Rating for shear in abutment cap with all 5 piles in sound condition

**(Reference AISC ASD 9th Ed. Beam Diagrams No. 39, pg 2-309.)**

### Shear in abutment cap

$$V_{DLSupC} := 0.607 W_{DLcap} \cdot (PileSpa)$$ **(Dead load shear at CL pile support)** $V_{DLSupC} = 6.34\,k$

$$V_{LLSupC} := 0.607 W_{LLcap} \cdot (PileSpa)$$ **(Live load shear at CL pile support)** $V_{LLSupC} = 13.73\,k$

**- Check shear at "d" from CL pile :**

$$x_{shear} := (PileSpa) - \frac{PileDiam}{2} - Depth_{abutcap} \qquad x_{shear} = 6.08\,ft$$

$$V_{DLSupd} := V_{DLSupC} \cdot \frac{x_{shear} - [0.393 \cdot (PileSpa)]}{(PileSpa) - [0.393 \cdot (PileSpa)]}$$ **(Dead load shear at "d" from pile support)** $V_{DLSupd} = 4.09\,k$

$$V_{LLSupd} := V_{LLSupC} \cdot \frac{x_{shear} - [0.393 \cdot (PileSpa)]}{(PileSpa) - [0.393 \cdot (PileSpa)]}$$ **(Live load shear at "d" from pile support)** $V_{LLSupd} = 8.86\,k$

## Calculate Inventory Rating Factor for Shear in Cap (5 piles in sound condition)

$$V_{RI} := \frac{2}{3} \cdot A_{Abutcap} \cdot F_{VInvCap} \qquad V_{RI} = 12.77\,k$$

$$RF_{IV} := \frac{V_{RI} - V_{DLSupd}}{V_{LLSupd}} \qquad RF_{IV} = 0.98$$

## Calculate Operating Rating Factor for Shear in Cap (5 piles in sound condition)

$$V_{RO} := \frac{2}{3} \cdot A_{Abutcap} \cdot F_{VOpCap} \qquad V_{RO} = 16.99\,k$$

$$RF_{OV} := \frac{V_{RO} - V_{DLSupd}}{V_{LLSupd}} \qquad RF_{OV} = 1.45$$

## Inventory Rating for Shear in Cap (5 piles in sound condition)

$Inv_{RatingV} := RF_{IV} \cdot 20$ HS $Inv_{RatingV} = 20$

## Operating Rating for Shear in Cap (5 piles in sound condition)

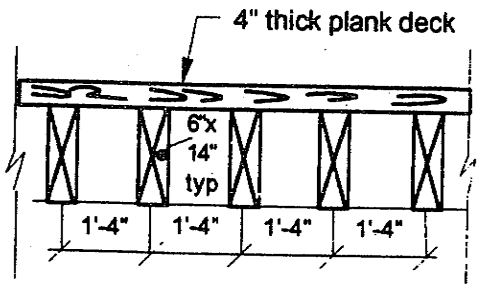
$Op_{RatingV} := RF_{OV} \cdot 20$ HS $Op_{RatingV} = 29$

2003 AASHTO "MCE"



EXAMPLE B3: - Timber Stringer

Given: A simple span, timber stringer bridge with a timber plank deck (two lanes). Span length - 17'-10" c-c bearings (field measured).




Partial X-Section
(nts)

Timber dimensions field measured (actual)

Good maintenance and inspection.

Smooth approaches, fair deck smoothness.

Year Built: 1930
Year Reconstructed: 1967

ADTT << 1000

Timber Species: Southern Pine No. 2

AASHTO Table 13.2.1A for No. 2:

$$F_b = 1200 \text{ psi}^* \text{ (new)}; \quad F_v = 90 \text{ psi (new)}$$

Load rate the 6"x14" stringers:

Dead Loads - Deck $\dfrac{(1'-4")4"}{144 \text{ in}^2/\text{ft}^2} \times 50 \text{ lbs/ft}^3 = 22.2 \text{ lbs/ft}$

Stringer $\dfrac{6"\text{x}14"}{144} \times 50 = \underline{29.2 \text{ lb/ft}}$

51.4 lb/ft say 0.055 k/ft

Live Load - Rate for H15 truck

Section Properties: (Again for stringers)

$$I_x = \frac{bh^3}{12} = \frac{6 \times 14^3}{12} = 1372 \text{ in}^4$$

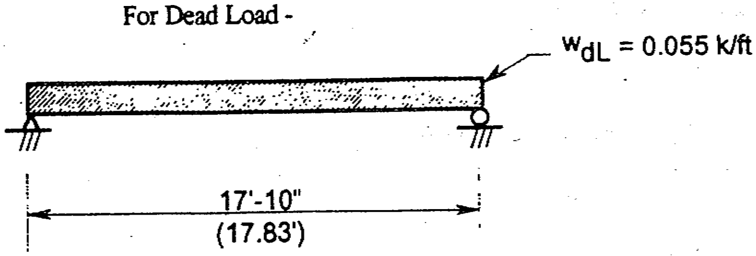
$$S_x = \frac{I_x}{h/2} = \frac{1372}{14/2} = 196 \text{ in}^3$$

$$A = bh = 6 \times 14 = 84 \text{ in}^2$$

* The provisions of AASHTO 13.3.7.1 should also be applied. For this sample $C_F = 1.0$ was assumed.

Midspan Moments:

For Dead Load -




$$M_D = \frac{w_{dL}\ L^2}{8} = \frac{0.055\ (17.83)^2}{8}$$

$$M_D = 2.19'^k$$

For H15 - From MANUAL Appendix A3, page 74[1]

| Span | $M_L$ |
|---|---|
| 17' | $51'^k$ |
| 18' | $54'^k$ |

← For 17.83' span, interpolate

$$M_L = 51 + \frac{17.83 - 17}{18\text{-}17}(54\text{-}51) = 53.5'^k$$

Allowable Stress Rating (MANUAL 6.4.1, 6.5.2 & 6.6.2)

(Consider stringer only; consider maximum moment and shear sections only for this example - see General Notes.)

Impact - MANUAL 6.7.4 use standard AASHTO

AASHTO 3.8.1.2 - No impact for timber members

$$I = 0$$

Distribution - MANUAL 6.7.3 indicate that standard AASHTO provisions may be used.

AASHTO 3.23.2.2 and Table 3.23.1
For two lanes and plank deck:

$$DF = \frac{S}{3.75} = \frac{16''/12''/\text{ft}}{3.75} = 0.36$$

Thus:

$$M_{LL+I} = M_L\ (1+I) \times DF = 53.5'^k \times (1+0) \times 0.36$$

$$M_{LL+I} = 19.26'^k$$

Stresses to be used:

Inventory: MANUAL 6.6.2.7(1) → use AASHTO

(1) Note the moment given in the MANUAL are for one line of wheels. The values given in AASHTO are for the entire axle and are therefore twice the MANUAL values.

For this specie $F_b = 1200$ psi

However since this bridge is more than 10 years old, AASHTO 13.2.4 applies.

$$F_b^{inv} = 0.9(1200)\,C_F = 1080 \text{ psi} = 1.08 \text{ ksi}$$

($C_F = 1.0$ (see note sh. 108))

and

$$F_v^{inv} = 90 \times 0.9 = 81 \text{ psi}$$

Operating: MANUAL 6.6.2.7(2)

$$F_b^{op} = F_b^{inv} \times 1.33 \times C_F = 1080 \times 1.33 \times 1.0$$

($C_F = 1.0$ (see note sh. 108))

$$F_b^{op} = 1436 \text{ psi say } 1440 \text{ psi} = 1.44 \text{ ksi}$$

and

$$F_v^{op} = 1.33\,F_v^{inv} = 1.33 \times 81 \text{ psi} = 108 \text{ psi}$$

<u>Inventory Level Rating</u>

Capacity: $M_{R_I} = F_b^{inv}\,S_x = 1.08 \text{ ksi} \times 196 \text{ in}^3 = 211.7 \text{ in-k}$

$M_{R_I} = 17.64$ ft-k

then

(MANUAL Eqn. 6-1a) $$RF_I^M = \frac{M_{R_I} - M_D}{M_{L+I}} = \frac{17.64'^k - 2.19'^k}{19.26'^k}$$

$$RF_I^M = \underline{\underline{0.80}} \text{ or } 0.80 \times 15 \text{ tons} = \underline{\underline{12 \text{ tons}}} \text{ H truck}$$

<u>Operating Level Rating</u>

Capacity: $M_{R_O} = F_b^{op}\,S_x = 1.44 \text{ ksi} \times 196 \text{ in}^3 = 282.2 \text{ in-k}$

$M_{R_O} = 23.52'^k$

then

(MANUAL Eqn. 6-1a) $$RF_O^M = \frac{M_{R_O} - M_D}{M_{L+I}} = \frac{23.52'^k - 2.19'^k}{19.26'^k}$$

$$RF_O^M = \underline{\underline{1.11}} \text{ or } 1.11 \times 15 \text{ tons} = \underline{\underline{16.6 \text{ tons}}} \text{ H truck}$$

Check Horizontal Shear

AASHTO 13.3.1 suggests that shear be computed at:

(1) A distance from the support equal to three times the depth of the stringer; or

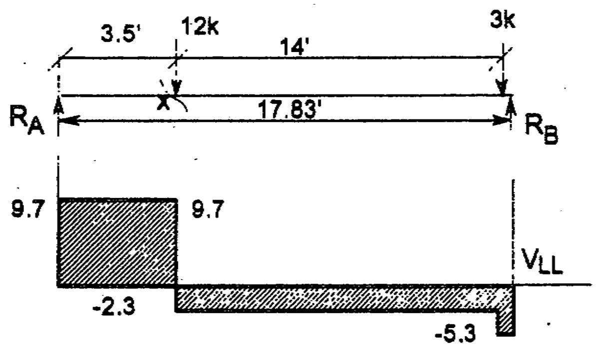
(2) At the quarter point, whichever is less.

Thus by: (1) $3(14") = 42" \leftarrow$ Controls = 3.5 ft

(2) $\frac{17.83' \times 12"/\text{ft}}{4} = 53.5"$

For H15 Truck: MANUAL, Appendix A8, pg. 79

$$V_x = \frac{15(x - 2.8)}{L}$$




where L = 17.83' x = 17.83 - 3.5 = 14.33

$$V_x = \frac{15(14.33 - 2.8)}{17.83} = 9.7 \text{ k},$$

per wheel line without distribution

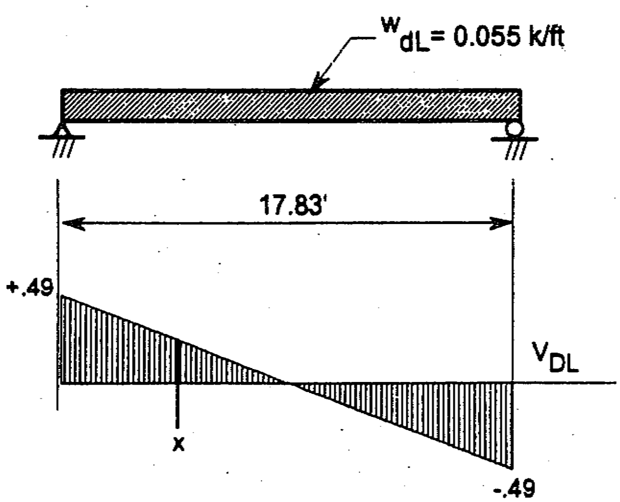
then per AASHTO 13.3.1

$$V_{Lx} = 1/2 \left[0.6\, V_x^{L\text{ no dist.}} + DF\, V_x^{L\text{no dist.}}\right]$$

$$V_{Lx} = 1/2\,[0.6\,(9.7) + 0.36\,(9.7)]$$

$$V_{Lx} = 4.7 \text{ k}$$

For $w_{dL} = 0.055$ k/ft




$R_A = R_B = 1/2\, w_{dL}\, L$

$= 1/2\,(0.055) \times 17.83$

$= 0.49$ k

$V_{Dx} = 0.49 - .055 \times 3.5$

$V_{Dx} = 0.3$ k

Rating Based on Shear

Inventory:

Capacity: AASHTO Eqn. 13-1 solve for $V_R$

$$V_R = \frac{2}{3} bd\, f_v$$

then

$$V_{R_I} - \frac{2}{3}(6)(14)\ 81\ \text{psi} = 4536\ \text{lbs.} = 4.54\ \text{k}$$

$$\text{(MANUAL Eqn. 6-1a): } RF_I^V = \frac{V_{R_I} - V_{D_X}}{V_{L_X}} = \frac{4.54\ \text{k} - 0.3\ \text{k}}{4.7\ \text{k}}$$

$$RF_I^V = \underline{\underline{0.90}} \text{ or } 0.90 \times 15 \text{ tons} = \underline{\underline{13.5 \text{ tons}}} \text{ H truck}$$

Operating:

Capacity:

$$V_{R_O} = \frac{2}{3}(6)(14)(108\ \text{psi}) = 6048\ \text{lbs.} = 6.05\ \text{k}$$

$$\text{(MANUAL Eqn. 6-1a): } RF_O^V = \frac{V_{R_O} - V_{D_X}}{V_{L_X}} = \frac{6.05\ \text{k} - 0.3\ \text{k}}{4.7\ \text{k}}$$

$$RF_O^V = \underline{\underline{1.22}} \text{ or } 1.22 \times 15 \text{ tons} = \underline{\underline{18.5 \text{ tons}}} \text{ H truck}$$

Load Factor Rating

Not currently available for timber.

Load and Resistance Factor Rating

Not currently available for timber.

SUMMARY OF RESULTS

| Method/Force | RF | H Truck Max. Load (tons) |
|---|---|---|
| Allowable Stress | | |
| Moment: | | |
| Inventory | 0.80 | 12.0 |
| Operating | 1.11 | 16.6 |
| Allowable Stress | | |
| Shear: | | |
| Inventory | 0.90 | 13.5 |
| Operating | 1.22 | 18.3 |

∴ Rating governed by moment rather than shear

2005 GUIDE "MCE" LRFR

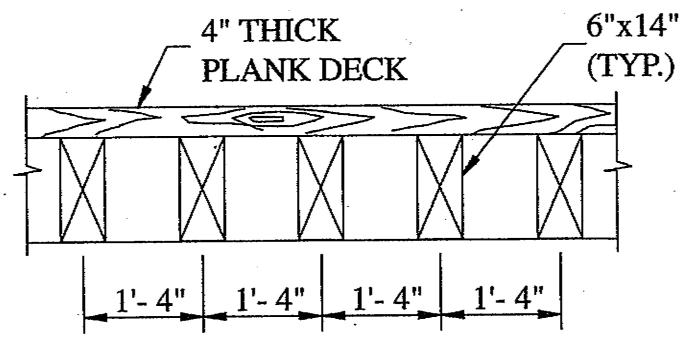


**Example A4:** **Timber Stringer Bridge**
**Evaluation of an Interior Stringer**

GIVEN:

| | |
|---|---|
| Span: | 17 ft. 10 in. |
| Year Built: | 1930 |
| Year Reconstructed: | 1967 |
| Material | Southern Pine No. 2 |
| Condition: | No deterioration. NBI Item 59 Code = 6 |
| Riding Surface: | Unknown condition |
| Traffic: | Two Lanes |
| *ADTT* (one direction): | 150 |
| Skew: | 0 degrees |

Note: Same bridge as in Example B3 1994 AASHTO MCE.




PARTIAL CROSS SECTION

I. CHECK MOMENT CAPACITY

DEAD LOAD ANALYSIS—INTERIOR STRINGER

1. Components and Attachments *DC*

Deck: $\frac{16}{12} \times \frac{4}{12} \times 0.050$ = 0.022 kip/ft. — LRFD Table 3-3

Stringer: $\frac{6 \times 14}{144} \times 0.050$ = 0.029 kip/ft.

Total per stringer = 0.051 kip/ft.

Say = 0.055 kip/ft.

$$M_{DC} = \frac{1}{8} \times 0.055 \times 17.83^2$$
$$= 2.19 \text{ kip-ft.}$$

2. Wearing Surface $DW = 0$

LIVE LOAD ANALYSIS—INTERIOR STRINGER

I. Distribution Factor for Moment and Shear

The stringers are continuously braced by the deck and diaphragms (6 in. x 14 in.) are present at the Bearings; $C_s = 1.0$

$$F_b = F_{bo}\, C_F\, C_M\, C_D$$

$F'_{bo} = 3.3$ ksi Southern Pine No. 2 — LRFD Table 8-1

Size Factor: $C_F = 1.02$ $6 \times 14$ stringers — LRFD Table 8-7
(value in LRFD Table 8-7 for Width = 14 in. and Thickness = 4 in.)

Moisture Content Factor: $C_M = 1.0$ — LRFD 8.4.4.3

Deck Factor: $C_D = 1.0$ Plank Deck — LRFD 8.4.4.4

$$F_b = (3.3)\,(1.02)\,(1.0)\,(1.0)$$
$$= 3.36 \text{ ksi}$$

Nominal Resistance: $M_n = F_b\, S\, C_s$

$$= (3.36)(196)(1.0)\left(\frac{1}{12}\right)$$
$$= 54.9 \text{ kip-ft.}$$

GENERAL LOAD RATING EQUATION — 6.4.2

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_p)(P)}{(\gamma_L)(LL+IM)}$$ Eq. (6-1)

EVALUATION FACTORS (For Strength Limit State) — LRFD 8.5.2.2

a) Resistance Factor $\varphi$
$\varphi = 0.85$ for Flexure
$\varphi = 0.75$ for Shear

b) Condition Factor $\varphi_c$ — 6.4.2.3
$\varphi_c = 1.0$ Good Condition

c) System Factor $\varphi_s$ — 6.4.2.4
$\varphi_s = 1.0$ for flexure and shear in timber bridges

1. **DESIGN LOAD RATING** — 6.4.3

A) Strength I Limit State — 6.7.4.1

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)}{(\gamma_L)(LL+IM)}$$

a) Inventory Level

Table 6-1

| LOAD | LOAD FACTOR |
|---|---|
| $DC$ | 1.25 |
| $LL$ | 1.75 |

$V_{LU}$ = maximum vertical shear at $3d$ or $L/4$ due to undistributed wheel loads (kips)

= For undistributed wheel loads, one line of wheels is assumed to be carried by one bending member. — LRFD 4.6.2.2.2a

$$= \frac{V_{TANDEM}}{2} = (34.6 \times 1.165)/2 = 20.15 \text{ kips}$$

$V_{LD}$ = maximum vertical shear at $3d$ or $L/4$ due to wheel loads distributed laterally as specified herein (kips)

$= V_{LL+IM} = 8.8$ kips

$$V_{LL} = 0.50[(0.60 \times 20.15) + 8.8] = 10.45 \text{ kips}$$

COMPUTE NOMINAL SHEAR RESISTANCE

$$V_n = \frac{F_v bd}{1.5}$$ LRFD Eq. (8-14)

$F_v = F_{vo}\ C_F\ C_M\ C_D$ — LRFD Eq. (8-1)

$F_{vo}$ = 0.300 ksi — Southern Pine No. 2 — LRFD Table 8-1

$C_F$ = 1.02

$C_M$ = 1.00

$C_D$ = 1.00

$F_v$ = (0.3) (1.02) (1.0) (1.0)

= 0.306 ksi

$$V_n = \frac{(0.306)(6)(14)}{1.5} = 17.1 \text{ kip}$$

a) Inventory Level

| LOAD | LOAD FACTOR |
|---|---|
| *DC* | 1.25 |
| *LL* | 1.75 |

Shear *RF*

$$= \frac{(1.0)(1.0)(0.75)(17.1) - (1.25)(0.426)}{(1.75)(10.45)}$$

$= 0.67$

b) Operating Level

Shear *RF*

$$= 0.67 \times \frac{1.75}{1.35} = 0.87$$

No service limit states apply.

Summary

| Truck: | Type 3 | Type 3S2 | Type 3-3 |
|---|---|---|---|
| Weight (tons) | 25 | 36 | 40 |
| *RF* | 1.09 | 1.19 | 1.32 |
| Safe Load Capacity (tons) | 27 | 43 | 53 |

**Summary of Rating Factors**

EXAMPLE A4

INTERIOR STRINGER

| Limit State | | Design Load Rating | | Legal Load Rating | | |
|---|---|---|---|---|---|---|
| | | Inventory | Operating | T3 | T3S2 | T3-3 |
| Strength I | Flexure | 0.55 | 0.71 | 1.09 | 1.19 | 1.32 |
| | Shear | 0.67 | 0.87 | 1.24 | 1.36 | 1.50 |

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

2-20-98

Sheet No. 1 of 1
Rated by JWD
Checked by
DATE 2-20-98

## TIMBER BEAM RATING SHEET

EXAMPLE

Side 1

| BRIDGE LOCATION AND DESCRIPTION | |
|---|---|
| Bridge No: 92841 | Description: 1 SPAN OVER STREAM |
| Route: 175 | |
| Roadway Width: 18' | Location: |
| Year Built: 1968 | |

Year Remodeled: — Slab Thickness: 5" PLANK W.C.Thickness: 3" BIT

Span Rated: 1 Span Length(L): 25' Beam Spacing (S): 2.25' Impact: 0

**SUMMARY OF RATING AND LOAD POSTING**

| INVENTORY RATING | OPERATING RATING | LOAD POSTING REQ'D ? | LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED) | | |
|---|---|---|---|---|---|
| | | | Vehicle Type M3 Weight = 24T | Semi-Trailer Comb. Type M3S2 Weight = 36T | Truck & full Trailer Type M3-3 Weight = 40T |
| HS 13.2 | HS 19.0 | YES ____ NO ✓ | ____ Tons | ____ Tons | ____ Tons |

**RATING DATA**

Dist. Factor (p. 32, AASHTO, 1997) = S/4 = .563

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Critical Point Location | Number | ℄ | ℄ |
| Section Modulus (-loss) | in.$^3$ | 423.7 | 423.7 |
| Allowable Stress (See reverse side) | Kips/in.$^2$ | fi = 1.465 | fo = 1.33 fi = 1.949 |
| Resisting Moment/BM = $\frac{SM(fi\ or\ fo)}{12}$ | Ft. Kips | 51.7 ① | 68.8 ① |
| Dead Load Moment/Bm | Ft. Kips | 13.2 ② | 13.2 ② |
| Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + Impact)(Dist. Factor)}$ | Ft. Kips | $\frac{51.7-13.2}{(1)(.563)} = 68.4$ ④ | 98.78 ④ |
| HS 20 Moment per wheel line(See Ill. #9) | Ft. Kips | 103.7 ⑤ | 103.7 ⑤ |
| HS Rating $\frac{④}{⑤}(20)$ | Number | HS 13.2 | HS 19.0 |

| DEAD LOAD DESCRIPTION | DL per ft. of Beam |
|---|---|
| Overburden = .25' x 2.25' x 144 #/C.F. | = 81.0 lb./ft. |
| Slab = 5", (50 #/C.F.)(2.25') | = 47.0 lb./ft. |
| Beam (Nominal Size = 6" x 22") SIZED = 5.5" x 21.5" x 50 #/CF / 144 | = 41.0 lb./ft. |
| Railing, Diaphragms, etc. = | = — lb./ft. |
| Total Dead Load = W | = 169 lb./ft. |
| Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = $\frac{169(25)^2}{8000}$ | = 13.2 Ft-Kips |

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

2-20-98

**TIMBER BEAM RATING SHEET** EXAMPLE

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Side 2
BR 92841

**ALLOWABLE STRESS FOR TIMBER BEAMS -- AASHTO 13.6.4.1**

$fi = Fb' = F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r$ For sawn lumber, only $C_M$, $C_D$ $C_F$ & $C_r$ apply.

Note: For Glulam timber beams, $C_F$ is not used. Use $C_V$ per Section 13.6.4.3.

$C_M$- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, $C_M$ = **0.85** unless Fb($C_F$) are less then 1150, then $C_M$ = 1.0.
(For glulam beam, from Table 13.5.3A, $C_M$ = 0.80)

$C_D$-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), $C_D$ = **1.15.**

$C_F$- Sect. 13.6.4.2 Lumber 2" to 4" thick use Table 13.5.1A. For lumber thicker then 5" $C_F = (12/d)^{1/9}$.
I.E. for 8 x 14 beam, $C_F = (12/14)^{1/9}$ = **0.983.**
$C_r$ -Table 13.5.1A. For lumber 2" to 4" thick, $C_r$ = 1.15. Otherwise $C_r$ = 1.0.

As an example, if $F_b$ = 1500psi, and beam is 8" x 14", then
fi = F'b = 1500(.85)(1.15)(0.983) =**1441 psi** allowable bending stress.
And fo = 1.33fi = **1917 psi**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 ( allowable Shear parallel to grain).
fv(inv) = F'v = $F_V$ $C_M$ $C_D$. If $F_V$ = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

<u>Use Table 3.23.1 for Live load distribution for bending analysis.</u>
<u>Use Section 13.6.5.2 for live load distribution for shear analysis</u>

ALLOWABLE STRESS FOR BENDING:

FROM PLAN; $Fb = 1600\,psi$, $CM = 0.85$, $CD = 1.15$, $C_F = \left(\frac{12}{21.5}\right)^{\frac{1}{9}} = .937$, $C_r = 1.0$

$\therefore Fb' = fi = 1600(.85)(1.15)(.937)(1.0) = \underline{1465\,psi}$ & $fo = 1.33 \times 1465 = \underline{1949\,psi}$

$SM = \frac{1}{6}(5.5)(21.5)^2 = \underline{423.7\,in^3}$

CHECK SHEAR: $V_{DL}(\text{at 'd' out}) = .169\left(\frac{25'}{2} - 1.79'\right) = 1.81^k$ $d = 1.79'$

$V_{LL}(\text{UNDIST}) = 16^k \frac{(19.13' + 5.63')}{25} = 16.16^k$

SECT. 13.6.5.2

$V_{LL}(\text{DIST.}) = 16.16 \times .563 = 9.1^k$ $V_{LL} = .5(.6 \times 16.16 + 9.1) = \underline{9.4^k}$

$\text{INV. RATING} = \frac{V_{CAP} - V_{DL}}{V_{LL}}(20)$ $V_{CAP} = \frac{2}{3} F'v\,(b)(d)$

$V_{CAP} = 109.25(5.5)(21.5)\frac{2}{3} = 8612^{\#} = 8.612^k$

$\therefore \text{INV. RATING(SHEAR)} = \frac{8.612 - 1.81}{9.4}(20) = \underline{HS\ 14.47}$ - NOT CRITICAL

DEPARTMENT OF TRANSPORTATION

Rated by ____________
Checked by ____________
DATE ____________

# TIMBER BEAM RATING SHEET

Side 1

## BRIDGE LOCATION AND DESCRIPTION

| | |
|---|---|
| Bridge No: | Description: |
| Route: | |
| Roadway Width: | Location: ____________ |
| Year Built: | ____________ |

Year Remodeled:____________ Slab Thickness:____________ W.C.Thickness:____________

Span Rated:____________ Span Length(L):____________ Beam Spacing (S):____________ Impact:____________

## SUMMARY OF RATING AND LOAD POSTING

| INVENTORY RATING | OPERATING RATING | LOAD POSTING REQ'D ? | LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED) | | |
|---|---|---|---|---|---|
| HS______ | HS______ | YES______ NO______ | Vehicle Type M3 Weight = 24T | Semi-Trailer Comb. Type M3S2 Weight = 36T | Truck & full Trailer Type M3-3 Weight = 40T |
| | | | ______Tons | ______Tons | ______Tons |

## RATING DATA

Dist. Factor (p. 32, AASHTO, 1997) = S/ = ____________

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Critical Point Location | Number | | |
| Section Modulus (-loss) | $in.^3$ | | |
| Allowable Stress (See reverse side) | $Kips/in.^2$ | fi = | fo = 1.33 fi = |
| Resisting Moment/BM = $\frac{SM(fi \text{ or } fo)}{12}$ | Ft. Kips | ① | ① |
| Dead Load Moment/Bm | Ft. Kips | ② | ② |
| Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + Impact)(Dist. Factor)}$ | Ft. Kips | ④ | ④ |
| HS 20 Moment per wheel line(See Ill. #9) | Ft. Kips | ⑤ | ⑤ |
| HS Rating $\frac{④}{⑤}(20)$ | Number | HS ______ | HS ______ |

| DEAD LOAD DESCRIPTION | DL per ft. of Beam |
|---|---|
| Overburden = | = lb./ft. |
| Slab = | = lb./ft. |
| Beam (Nominal Size = ) | = lb./ft. |
| Railing, Diaphragms, etc. = | = lb./ft. |
| Total Dead Load = W | = lb./ft. |
| Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = | = Ft-Kips |

STATE OF MINNESOTA
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2-20-98

Rated by
Checked by
DATE
Side 2

**TIMBER BEAM RATING SHEET**

**ALLOWABLE STRESS FOR TIMBER BEAMS -- AASHTO 13.6.4.1**

$fi = Fb' = F_b C_M C_D C_F C_V C_L C_f C_{fu} C_r$   For sawn lumber, only $C_M$, $C_D$ $C_F$ & $C_r$ apply.

Note: For Glulam timber beams, $C_F$ is not used. Use $C_V$ per Section 13.6.4.3.

$C_M$- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, $C_M$ = **0.85** unless $Fb(C_F)$ are less then 1150, then $C_M = 1.0$.
(For glulam beam, from Table 13.5.3A, $C_M = 0.80$)

$C_D$-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), $C_D$ = **1.15**.

$C_F$- Sect. 13.6.4.2 Lumber 2" to 4" thick use Table 13.5.1A. For lumber thicker then 5" $C_F = (12/d)^{1/9}$.
I.E. for 8 x 14 beam, $C_F = (12/14)^{1/9}$ = **0.983.**
$C_r$ -Table 13.5.1A. For lumber 2" to 4" thick, $C_r = 1.15$. Otherwise $C_r = 1.0$.

As an example, if $F_b$ = 1500psi, and beam is 8" x 14", then
fi = F'b = 1500(.85)(1.15)(0.983) =**1441 psi** allowable bending stress.
And fo = 1.33fi = **1917 psi**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 ( allowable Shear parallel to grain).
$fv(inv) = F'v = F_V C_M C_D$. If $F_V$ = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

<u>Use Table 3.23.1 for Live load distribution for bending analysis.</u>
<u>Use Section 13.6.5.2 for live load distribution for shear analysis</u>

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

2-20-98

Sheet No. 1 of 1
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Date 2-20-98
Side 1

**LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET**

EXAMPLE
NAILED PANEL

## BRIDGE LOCATION AND DESCRIPTION

| | |
|---|---|
| Bridge No: 03506 | Description: 3 SPANS OVER OTTER TAIL RIVER |
| Route: TWN 300 | |
| Roadway Width: 28' | Location: 1.4 M. N.E of Jct CSAH 29 |
| Year Built: 1981 | |

Year Remodeled: — Slab Thickness: 3"x14" LAM. W.C.Thickness: 3" Asphalt

Span Rated: ______ Span Length(L): 31'-2" Panel Width (Wp): 6'-0" Impact: 0

## SUMMARY OF RATING AND LOAD POSTING

| INVENTORY RATING | OPERATING RATING | LOAD POSTING REQ'D ? | LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED) | | |
|---|---|---|---|---|---|
| | | | Vehicle Type M3 Weight = 24T | Semi-Trailer Comb. Type M3S2 Weight = 36T | Truck & full Trailer Type M3-3 Weight = 40T |
| HS 27.1 | HS 38.8 | YES ____ NO ✓ | ______ Tons | ______ Tons | ______ Tons |

## RATING DATA

Dist. Factor (See Reverse Side) = D.F. = 1.5

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Critical Point Location | Number | ℄ | ℄ |
| Section Modulus (~~-loss~~) $\frac{1}{6}72(14)^2$ | in.$^3$ | 2352 | 2352 |
| Allowable Stress (See reverse side) | Kips/in.$^2$ | fi = 1.517 | fo = 1.33 fi = 2.018 |
| Resisting Moment/BM = $\frac{\text{SM(fi or fo)}}{12}$ | Ft. Kips | 297.3 ① | 395.5 ① |
| Dead Load Moment/panel | Ft. Kips | 70.0 ② | 70.0 ② |
| Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + \text{Impact})(\text{Dist. Factor})}$ | Ft. Kips | $\frac{297.3-70}{1(1.5)}$ = 151.55 ④ | 217.0 ④ |
| HS 20 Moment per wheel line(See Ill. #9) | Ft. Kips | 111.8 ⑤ | 111.8 ⑤ |
| HS Rating $\frac{④}{⑤}$(20) | Number | HS 27.1 | HS 38.8 |

## DEAD LOAD DESCRIPTION

| DEAD LOAD DESCRIPTION | DL per ft. of panel. |
|---|---|
| Overburden = .25' x 6' x 144 #/CF | = 216 lb./ft. |
| Lam. Deck (Nominal Size = 6' x 14") 6' x 1.17' x 50 #/CF | = 351 lb./ft. |
| Railing, Diaphragms, etc. = Sp. B7: .5 x .83 x 6 x 50 / 15.6 | = 8 lb./ft. |
| Total Dead Load = W | = 575 lb./ft. |
| Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = $\frac{.575(31.17)^2}{8}$ | = 70 Ft-Kips |

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

2-20-98

NAILED PANEL EXAMPLE

Rated by JWO
Checked by
Date: 2-20-98
Side 2

## LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET BR03506

### LIVE LOAD DISTRIBUTION FACTOR:

Determine the fraction of wheel load applied to each panel.
No longitudinal distribution of wheel loads.
Lateral distribution is as follows:

**Continuous Nailed Panels: AASHTO Section 3.25.2.2.**

Dist. Factor/Wh. = Wp/wheel dist. width = $\frac{6.0}{4.0} = 1.5$

Wheel dist. width = Tire width + 2(t). Tire width usually can be taken as 20". "t" = panel thickness. = 14"

∴ Wh DIST = 20" + 2(14) = 48"

Wp = panel width PANEL 'C' = 6'-0"

**Glue Lam. Panels: AASHTO Section 3.25.3.1.**

2 lanes: D.F. = $\frac{Wp}{3.75 + L/28}$ or $\frac{Wp}{5.00}$ : Use Greater, D. F. =

1 lane: D.F. = $\frac{Wp}{4.25 + L/28}$ or $\frac{Wp}{5.50}$ : Use Greater, D. F. =

D.F. = Lateral Live Load distribution factor.

### ALLOWABLE STRESS FOR TIMBER DESIGN – AASHTO Section 13.6.4.1

$fi = Fb' = F_bC_MC_DC_FC_VC_LC_fC_{fu}C_r$ For sawn lumber and glulam, only $C_M$, $C_D$, $C_F$ & $C_r$ apply.

$C_M$- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, $C_M$ = **0.85** for $F_b$ unless $F_b(C_F)$are less then 1150, then $C_M$ = 1.0.
(For glulam, Table 13.5.3A, $C_M$ = 0.80)

$C_D$-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), $C_D$ = **1.15**.

$C_F$- Sect. 13.6.4.2: For lumber 2" to 4" thick use Table 13.5.1A.
I.E. for 3 x 14 nail lam., $C_F$ = **0.9.**
$C_r$-Table 13.5.1A. For lumber 2" to 4" thick, $C_r$ = 1.15

As an example, if Fb = 1500 psi, then
fi = F'b = 1500(.85)(1.15)(0.9)(1.15) =**1517 psi** allowable bending stress.
And fo = 1.33fi = **2018 psi.**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 ( allowable Shear parallel to grain).
fv(inv) = F'v = $F_V$ $C_M$ $C_D$. If $F_V$ = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

FOR THIS EXAMPLE, fi & fo ARE AS SHOWN ABOVE.

SHEAR CHECK: $V_{DL}$ (at 'd' out) = .575$\left(\frac{31.17}{2} - 1.17\right)$ = 8.29^k/6' Panel, 'd' = 1.17

$V_{LL}$ (UNDIST) = 16^k(17.67' + 3.67')/31.17' = 10.95^k (SHEAR FOR LL @ 3('d') out)

$V_{LL}$ (DIST) = 10.95 × 1.5 = 16.42^k ∴ $V_{LL}$ = .5(.6 × 10.95 + 16.42) = 11.5^k/6' Panel

INV. RATING = $\frac{V_{CAP} - V_{DL}}{V_{LL}}(20)$. $V_{CAP} = \frac{2}{3}Fv(b)(d) = \frac{2}{3}\frac{109.25}{1000}(72)(14) = 73.4^k$

∴ INV. RATING (SHEAR) = $\frac{73.4 - 8.29}{11.5}(20)$ = HS 113 - NOT CRITICAL

DEPARTMENT OF TRANSPORTATION

Rated by ______
Checked by ______
Date ______
Side 1

# LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

## BRIDGE LOCATION AND DESCRIPTION

| | |
|---|---|
| Bridge No: | Description: |
| Route: | |
| Roadway Width: | Location: ______ |
| Year Built: | ______ |

Year Remodeled:______ Slab Thickness:______ W.C.Thickness:______

Span Rated:______ Span Length(L):______ Panel Width (Wp):______ Impact:______

## SUMMARY OF RATING AND LOAD POSTING

| INVENTORY RATING | OPERATING RATING | LOAD POSTING REQ'D ? | LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED) | | |
|---|---|---|---|---|---|
| HS______ | HS______ | YES______ NO______ | Vehicle Type M3 Weight = 24T | Semi-Trailer Comb. Type M3S2 Weight = 36T | Truck & full Trailer Type M3-3 Weight = 40T |
| | | | ______Tons | ______Tons | ______Tons |

## RATING DATA

Dist. Factor (See Reverse Side) = D.F. = ______

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Critical Point Location | Number | | |
| Section Modulus (-loss) | in. $^3$ | | |
| Allowable Stress (See reverse side) | Kips/in.$^2$ | fi = | fo = 1.33 fi = |
| Resisting Moment/BM = $\frac{SM(fi\ or\ fo)}{12}$ | Ft. Kips | ① | ① |
| Dead Load Moment/panel | Ft. Kips | ② | ② |
| Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + Impact)(Dist.\ Factor)}$ | Ft. Kips | ④ | ④ |
| HS 20 Moment per wheel line(See Ill. #9) | Ft. Kips | ⑤ | ⑤ |
| HS Rating $\frac{④}{⑤}(20)$ | Number | HS ______ | HS ______ |

| DEAD LOAD DESCRIPTION | DL per ft. of panel. |
|---|---|
| Overburden = | = lb./ft. |
| Lam. Deck (Nominal Size = ) | = lb./ft. |
| Railing, Diaphragms, etc. = | = lb./ft. |
| Total Dead Load = W | = lb./ft. |
| Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = | = Ft-Kips |

# ONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

**LIVE LOAD DISTRIBUTION FACTOR:**

Determine the fraction of wheel load applied to each panel.
No longitudinal distribution of wheel loads.
Lateral distribution is as follows:

**Continuous Nailed Panels: AASHTO Section 3.25.2.2.**

Dist. Factor/Wh. = Wp/wheel dist. width = ________________

Wheel dist. width = Tire width + 2(t). Tire width usually can be taken as 20". "t" = panel thickness.

Wp = panel width

**Glue Lam. Panels: AASHTO Section 3.25.3.1.**

2 lanes: D.F. = $\frac{Wp}{3.75 + L/28}$ or $\frac{Wp}{5.00}$ : Use Greater, D. F. =

1 lane: D.F. = $\frac{Wp}{4.25 + L/28}$ or $\frac{Wp}{5.50}$ : Use Greater, D. F. =

D.F. = Lateral Live Load distribution factor.

## ALLOWABLE STRESS FOR TIMBER DESIGN -- AASHTO Section 13.6.4.1

$fi = Fb' = F_bC_MC_DC_FC_VC_LC_fC_{fu}C_r$ For sawn lumber and glulam, only $C_M$, $C_D$, $C_F$ & $C_r$ apply.

$C_M$- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, $C_M$ = **0.85** for $F_b$ unless $F_b(C_F)$are less then 1150, then $C_M$ = 1.0.
(For glulam, Table 13.5.3A, $C_M$ = 0.80)

$C_D$-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), $C_D$ = **1.15**.

$C_F$- Sect. 13.6.4.2: For lumber 2" to 4" thick use Table 13.5.1A.
I.E. for 3 x 14 nail lam., $C_F$ = **0.9.**
$C_r$-Table 13.5.1A. For lumber 2" to 4" thick, $C_r$ = 1.15

As an example, if Fb = 1500 psi, then
fi = F'b = 1500(.85)(1.15)(0.9)(1.15) =**1517 psi** allowable bending stress.
And fo = 1.33fi = **2018 psi**.

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 ( allowable Shear parallel to grain).
fv(inv) = F'v = $F_V$ $C_M$ $C_D$. If $F_V$ = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi**.

2/16/94

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

Sheet No. 1 of 1
Rated By JDL
Checked By
Date 4/30/94

# LOGITUDINAL LAMINATED TIMBER DECK RATING SHEET

## BRIDGE LOCATION AND DESCRIPTION

Bridge No. 31531
Route Co. RD. 253
Rdwy. Width 28'-1"
Span Length 22'-6"
Year Built 1994
Year Remodeled —

Description 2 Span Nail Laminated Timber Panel Bridge, Interior Panel
Location Co. Rd. 253 over the Little Bowstring River, Itasca Co.

Slab Thickness 14" W.C. Thickness 4"

## SUMMARY OF RATING AND LOAD POSTING

| Inventory Rating | Operating Rating | Load Posting Req'd ? | LOAD POSTING LIMITS ⓐ (Complete when load posting is required) | | |
|---|---|---|---|---|---|
| | | | Vehicle Type M3 Weight=24T | Semi-Trailer Comb. Type M3S2 Weight=36T | Truck & Full Trailer Type M3-3 Weight=40T |
| H ____ or HS 37.8 | HS 52.2 | ____ Yes<br>X No | ____ Tons | ____ Tons | ____ Tons |

## RATING DATA

Dist. factor (See Reverse Side) = 1.58 Beam Spacing = 6'-4"

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Critical Point Location | Number | ℄ | ℄ |
| Impact (I = 0 for timber S.S.) | Number | 0 | 0 |
| Section Modulus (-loss) $\frac{1}{6} * 76" * 14^2 in^2$ | $In.^3$ | 2482.7 see Plan see Note #6 | 2482.7 |
| Allowable Stress | $Kips/In.^2$ | fi = ~~1.600~~ 1.5 | fo = 1.33 fy = ~~2.128~~ 1.995 |
| Resisting Moment/BM = $\frac{SM(fi \text{ or } fo)}{12}$ | Ft. Kips | 310.3 ① | 412.7 ① |
| ⓑ Dead Load Moment/BM | Ft. Kips | 41.0 ② | 41.0 ② |
| Mom. Avail. for LL/Wh. Line = $\frac{① - ②}{(1+Impact)(Dist.Factor)}$ | Ft. Kips | 170.4 ④ | 235.2 ④ |
| ⓒ HS 20 Moment per wheelline | Ft. Kips | 90.0 ⑤ | 90.0 ⑤ |
| HS Rating $\frac{④}{⑤}$ (20) | Number | HS 37.8 | HS 52.2 |

ⓐ Load Posting : For simple spans see illustration #'s 3, 4 & 10
ⓑ Dead Load Moment: For Simple Spans, DL Mom. = $\frac{1}{8} \frac{W(L)^2}{1000} = \frac{1}{8} * \frac{647.4 \frac{lb}{ft} * 22.5^2 ft^2}{1000} = 41.0'\text{-}k$
ⓒ HS 20 Moment: For Simple Spans see illustration #9

## DEAD LOAD DESCRIPTION

| | | DL per ft. of Beam |
|---|---|---|
| Overburden $4"$ asphalt $= 9\ lb/sq.ft\text{-}in * 4 in * (6'\text{-}4")$ | = | 228 lb./ft. |
| Lam. Deck (Size = 14" x 6'-4") $14" * 1 ft/12 in * (6'\text{-}4") * 50\ lb/ft^3$ | = | 369.4 lb./ft. |
| Railing, Diaphragms, etc. Rail to ext. panels, Say misc. = 50 lb/ft | = | 50.0 lb./ft. |
| Total Dead Load = W | = | 647.4 lb./ft. |

17

3/9/94

Live Load Distribution Factor:

No Longitudinal Distribution Of Wheel Loads.

Lateral Distribution Is As Follows.

Continuous Nailed Panels: AASHTO, 3.25.2.2

$$\text{Dist. Factor/Wh} = \frac{W_p}{S} = \frac{6'\text{-}4''}{\frac{20''}{12}+\frac{2(14)''}{12}} = 1.58$$

S = Tire Width + 2(t), Tire Width = 20", t = Panel Thickness

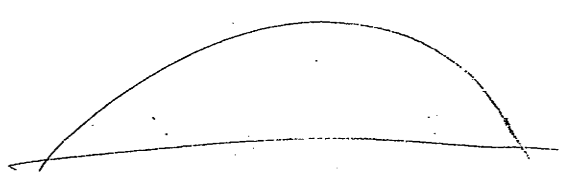
$W_p$ = Panel ~~Thickness~~ Width

Glue Lam. Panels: AASHTO, 3.25.3.1

2 Lanes: $D.F. = \frac{W_p}{3.75+\frac{L}{28}}$ or $\frac{W_p}{5.00}$ : Use Greater, D.F.=

1 Lane: $D.F. = \frac{W_p}{4.25+\frac{L}{28}}$ or $\frac{W_p}{5.50}$ : Use Greater, D.F.=

D.F. = Lateral Live Load Distribution Factor.



STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

2-20-98

Sheet No. 1 of 1
Rated by JWD
Checked by
Date 2-20-98
Side 1

# LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

EXAMPLE
GLULAM DECK

## BRIDGE LOCATION AND DESCRIPTION

| | |
|---|---|
| Bridge No: 10000 | Description: 1 SPAN OVER STREAM |
| Route: — | |
| Roadway Width: 32' | Location: 2.0 M. W. of Jct. CSAH 4 |
| Year Built: 1990 | |

Year Remodeled: — Slab Thickness: 3"x8¾" LAM. W.C.Thickness: 3" Asphalt

Span Rated: ______ Span Length(L): 20' Panel Width (Wp): 4'0" Impact: 0

## SUMMARY OF RATING AND LOAD POSTING

| INVENTORY RATING | OPERATING RATING | LOAD POSTING REQ'D ? | LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED) | | |
|---|---|---|---|---|---|
| | | | Vehicle Type M3 Weight = 24T | Semi-Trailer Comb. Type M3S2 Weight = 36T | Truck & full Trailer Type M3-3 Weight = 40T |
| HS 20.8 | HS 29.0 | YES ____ NO ✓ | — Tons | — Tons | — Tons |

## RATING DATA

Dist. Factor (See Reverse Side) = D.F. = .896

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Critical Point Location | Number | ℄ | ℄ |
| Section Modulus (-loss) $\frac{1}{6}(48)(8.75)^2$ | in.$^3$ | 612.5 | 612.5 |
| Allowable Stress (See reverse side) | Kips/in.$^2$ | fi = 1.745 | fo = 1.33 fi = 2.32 |
| Resisting Moment/BM = $\frac{SM(fi\ or\ fo)}{12}$ | Ft. Kips | 89.07 ① | 118.41 ① |
| Dead Load Moment/panel | Ft. Kips | 14.5 ② | 14.5 ② |
| Mom. Avail. for LL/ Wh. Line = $\frac{①\ -\ ②}{(1 + Impact)(Dist. Factor)}$ | Ft. Kips | $\frac{89.07 - 14.5}{.896}$ = 83.2 ④ | 116.0 ④ |
| HS 20 Moment per wheel line(See Ill. #9) | Ft. Kips | 80.0 ⑤ | 80.0 ⑤ |
| HS Rating $\frac{④}{⑤}$(20) | Number | HS 20.8 | HS 29.0 |

## DEAD LOAD DESCRIPTION

| | DL per ft. of panel. |
|---|---|
| Overburden = .25' x 4' x 144 #/CF | = 144 lb./ft. |
| Lam. Deck (Nominal Size = 8¾" x 4') (8.75x4x150 #/CF)/12 | = 146 lb./ft. |
| Railing, Diaphragms, etc. = taken by Exterior Panel | = — lb./ft. |
| Total Dead Load = W | = 290 lb./ft. |
| Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = $\frac{290(20)^2}{8000}$ = | = 14.5 Ft-Kips |

STATE OF MINNESOTA 2-20-98
DEPARTMENT OF TRANSPORTATION

Rated by JWD
Checked by
Date: 2-20-98
Side 2

GLULAM DECK EXAMPLE

## LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

BR 10,000

### LIVE LOAD DISTRIBUTION FACTOR:

Determine the fraction of wheel load applied to each panel.
No longitudinal distribution of wheel loads.
Lateral distribution is as follows:

**Continuous Nailed Panels: AASHTO Section 3.25.2.2.**

Dist. Factor/Wh. = Wp/wheel dist. width = ______________

Wheel dist. width = Tire width + 2(t). Tire width usually can be taken as 20". "t" = panel thickness.

Wp = panel width

**Glue Lam. Panels: AASHTO Section 3.25.3.1.**

2 lanes: D.F. = $\frac{Wp}{3.75 + L/28}$ or $\frac{Wp}{5.00}$ : Use Greater, D. F. = $\frac{4}{3.75 + \frac{20}{28}}$ = .896 or $\frac{4}{5}$ = .80
↑ CRITICAL

1 lane: D.F. = $\frac{Wp}{4.25 + L/28}$ or $\frac{Wp}{5.50}$ : Use Greater, D. F. = ∴ DF = .896

D.F. = Lateral Live Load distribution factor.

### ALLOWABLE STRESS FOR TIMBER DESIGN -- AASHTO Section 13.6.4.1

fi = Fb' = $F_bC_MC_DC_FC_VC_LC_fC_{fu}C_r$ For sawn lumber and glulam, only $C_M$, $C_D$, $C_F$ & $C_r$ apply.

$C_M$- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, $C_M$ = **0.85** for $F_b$ unless $F_b(C_F)$are less then 1150, then $C_M$ = 1.0.
(For glulam, Table 13.5.3A, $C_M$ = 0.80)

$C_D$-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), $C_D$ = **1.15**.

$C_F$- Sect. 13.6.4.2: For lumber 2" to 4" thick use Table 13.5.1A.
I.E. for 3 x 14 nail lam., $C_F$ = **0.9**.
$C_r$-Table 13.5.1A. For lumber 2" to 4" thick, $C_r$ = 1.15

As an example, if Fb = 1500 psi, then
fi = F'b = 1500(.85)(1.15)(0.9)(1.15) =**1517 psi** allowable bending stress.
And fo = 1.33fi = **2018 psi.**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 ( allowable Shear parallel to grain).
fv(inv) = F'v = $F_V$ $C_M$ $C_D$. If $F_V$ = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

EXAMPLE: Fb = 1500 psi, CM = 0.80, CD = 1.15, CF (for 3"x8¾") = 1.1, CR = 1.15
∴ fi = F'b = 1500(.8)(1.15)(1.1)(1.15) = 1745 psi, fo = 1.33×1745 = 2320 psi

CHECK SHEAR: VDL (at 'd' out) = .29($\frac{20}{2}$ − .75) = 2.68^k / 4' PANEL
VLL (UNDIST) = 16^k(17.75' + 3.75')/20' = 17.2^k, VLL (DIST) = 17.2×.896 = 15.4^k
Sect 13.6.5.2: VLL = .5(.6×17.2 HS.4) = 12.86^k / 4' PANEL
INV. RATING = $\frac{VCAP - VDL}{VLL}$(20), VCAP = $\frac{2}{3}$ F'v (b)(d) = $\frac{2}{3}$ $\frac{109.25}{1000}$(48)(8.75) = 30.59^k
∴ INV RATING (SHEAR) = $\frac{30.59 - 2.68}{12.86}$(20) = HS 43.4 - NOT CRITICAL

DEPARTMENT OF TRANSPORTATION

# LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

Rated by ______________
Checked by ______________
Date ______________
Side 1

## BRIDGE LOCATION AND DESCRIPTION

| | |
|---|---|
| Bridge No: | Description: |
| Route: | |
| Roadway Width: | Location: ______________ |
| Year Built: | ______________ |

Year Remodeled: ______________ Slab Thickness: ______________ W.C.Thickness: ______________

Span Rated: ______________ Span Length(L): ______________ Panel Width (Wp): ______________ Impact: ______________

## SUMMARY OF RATING AND LOAD POSTING

| INVENTORY RATING | OPERATING RATING | LOAD POSTING REQ'D ? | LOAD POSTING LIMITS (See Ill. 3 or 4, & 10) (COMPLETE WHEN LOAD POSTING IS REQUIRED) | | |
|---|---|---|---|---|---|
| HS______ | HS______ | YES______ NO______ | Vehicle Type M3 Weight = 24T | Semi-Trailer Comb. Type M3S2 Weight = 36T | Truck & full Trailer Type M3-3 Weight = 40T |
| | | | ______Tons | ______Tons | ______Tons |

## RATING DATA

Dist. Factor (See Reverse Side) = D.F. = ______________

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Critical Point Location | Number | | |
| Section Modulus (-loss) | $in.^3$ | | |
| Allowable Stress (See reverse side) | $Kips/in.^2$ | fi = | fo = 1.33 fi = |
| Resisting Moment/BM = $\frac{SM(fi\ or\ fo)}{12}$ | Ft. Kips | ① | ① |
| Dead Load Moment/panel | Ft. Kips | ② | ② |
| Mom. Avail. for LL/ Wh. Line = $\frac{① - ②}{(1 + Impact)(Dist.\ Factor)}$ | Ft. Kips | ④ | ④ |
| HS 20 Moment per wheel line(See Ill. #9) | Ft. Kips | ⑤ | ⑤ |
| HS Rating $\frac{④}{⑤}(20)$ | Number | HS ______ | HS ______ |

| DEAD LOAD DESCRIPTION | DL per ft. of panel. |
|---|---|
| Overburden = | = lb./ft. |
| Lam. Deck (Nominal Size = ) | = lb./ft. |
| Railing, Diaphragms, etc. = | = lb./ft. |
| Total Dead Load = W | = lb./ft. |
| Dead Load Moment = $\frac{(W)(L)^2}{(8)(1000)}$ = | = Ft-Kips |

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

2-20-98

Rated by
Checked by
Date:
Side 2

# LONGITUDINAL LAMINATED TIMBER DECK RATING SHEET

**LIVE LOAD DISTRIBUTION FACTOR:**

Determine the fraction of wheel load applied to each panel.
No longitudinal distribution of wheel loads.
Lateral distribution is as follows:

**Continuous Nailed Panels: AASHTO Section 3.25.2.2.**

Dist. Factor/Wh. = Wp/wheel dist. width = ______________

Wheel dist. width = Tire width + 2(t). Tire width usually can be taken as 20". "t" = panel thickness.

Wp = panel width

**Glue Lam. Panels: AASHTO Section 3.25.3.1.**

2 lanes: $D.F. = \frac{Wp}{3.75 + L/28}$ or $\frac{Wp}{5.00}$ : Use Greater, D. F. =

1 lane: $D.F. = \frac{Wp}{4.25 + L/28}$ or $\frac{Wp}{5.50}$ : Use Greater, D. F. =

D.F. = Lateral Live Load distribution factor.

**ALLOWABLE STRESS FOR TIMBER DESIGN -- AASHTO Section 13.6.4.1**

$fi = Fb' = F_bC_MC_DC_FC_VC_LC_fC_{fu}C_r$ For sawn lumber and glulam, only $C_M$, $C_D$, $C_F$ & $C_r$ apply.

$C_M$- Sect. 13.5.5.1: Expect all bridge timbers to exceed 19% moisture content.
From Table 13.5.1A, $C_M$ = **0.85** for $F_b$ unless $F_b(C_F)$are less then 1150, then $C_M$ = 1.0.
(For glulam, Table 13.5.3A, $C_M$ = 0.80)

$C_D$-Sect. 13.5.5.2: See table 13.5.5A, page 335. Use 2 months (Vehicle LL), $C_D$ = **1.15**.

$C_F$- Sect. 13.6.4.2: For lumber 2" to 4" thick use Table 13.5.1A.
I.E. for 3 x 14 nail lam., $C_F$ = **0.9.**
$C_r$-Table 13.5.1A. For lumber 2" to 4" thick, $C_r$ = 1.15

As an example, if Fb = 1500 psi, then
fi = F'b = 1500(.85)(1.15)(0.9)(1.15) =**1517 psi** allowable bending stress.
And fo = 1.33fi = **2018 psi.**

Shear rating may be checked, see section 13.6.5.2.
Allowable Shear Stress Sect. 13.6.5.3 ( allowable Shear parallel to grain).
fv(inv) = F'v = $F_V$ $C_M$ $C_D$. If $F_V$ = 95 psi, then F'v = 95(1.0)(1.15) = **109.25 psi.**

EXAMPLE

2-20-98

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

RATING SHEET FOR
TRANSVERSE TIMBER PLANK DECK

Sheet No. 1
Rated by JWP
Checked by
Date 2-20-98
Side 1

## BRIDGE LOCATION AND DESCRIPTION

Bridge No. 92888
Route 175
Year Built 1968
Year Remodeled

Long. Beam Spacing ( C/C) 3.0
Transverse Plank Thickness (t) 3"
Transverse Plank Width (b) 12"

## SUMMARY OF RATING AND LOAD POSTING

| Inventory rating | Operating Rating | Load Posting Req'd? | LOAD POSTING LIMITS @ (Complete when load posting is required) | | |
|---|---|---|---|---|---|
| | | | Vehicle Type M3 Weight = 24T | Semi-trailer Comb. Type M3S2 Weight = 36T | Truck & Full Trailer Type M3S3 Weight = 40T |
| H S 7.9 | HS 10.5 | ✓ Yes<br>___ No | 22 Tons | 34 Tons | 34 Tons |

## RATING FROM WORK SHEET TABLE OR CALCULATIONS BELOW

(See back side for explanation)

S = C/C - 0.23 = 2.77 (ft) No Impact

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Allowable Stress (assumed) | kips/sq.in | $f_i$ = ~~1.50~~ 1.64 | $f_o$ = 1.33 fi = ~~2.0~~ 2.18 |
| Mom Avail. For LL/Plank = SM(fi or fo)/12 -.01 SM = 18 in$^3$ | Ft. Kips | 2.45 ① | 3.26 ① |
| M (HS) = 12.8 ( S/4 - .2083 ) | Ft. Kips | 6.2 ② | 6.2 ② |
| RATING = $\frac{①}{②}$ (20) | number | HS 7.9 | HS 10.5 |

**POSTING:** M ( LEGAL) = .5625M (HS) = 3.49

Posting Limits:

Vehicle (M3) = $\frac{\text{Mom Avail(Oper)}\ 3.26}{\text{M (legal)}\ 3.49}$ (24) = 22.4 tons

Combination Vehicles (M3S2 or M3S3 ) = $\frac{\text{Mom Avail(Oper)}\ 3.26}{\text{M( legal)}\ 3.49}$ (36.65) = 34.2 Tons

EXAMPLE
PLAN Fb = 1.4 ksi



## WORK SHEET EXPLANATION

**S = C/C - 0.23: See AASHTO Sect. 3.25.1.2. Assume flange width = 5.5", then S = C/C - 5.5/2x12 = C/C - 0.23.**

**Allowable Stress: For older timber planks where timber stress is unknown, use fi = 1.5 ksi & fo = 2.0 ksi. If timber bending stress is known, see AASHTO 13.6.4.1. For 2" to 4" thick planks, Fb' = Fb,$C_M$,$C_D$,$C_{fu}$ and if $C_M$ =0.85, $C_D$ = 1.15, $C_{fu}$ = 1.2, then Fb' = Fb (1.17). Example: 3" x 12" plank, Fb (plan) = 1.4 ksi, then fi = 1.4(1.17) = $\boxed{1.64 \text{ ksi}}$ and fo = 1.33 x 1.64 = $\boxed{2.18 \text{ ksi.}}$**

**Moment available for LL/plank: Assume dead load moment of 0.01 ft-kip, then Mom avail. = SM(fi or fo)/12 - .01 where SM = (b)$(t)^2$/6. Example: 3" x 12" plank, fi = 1.5 ksi, then SM = (12 x 3 x 3)/6 = 18 in cu. and Mom avail. (Inv) = 18 x 1.5/12 - .01 = 2.24 ft kip/plank.**

**M ($_{HS}$) = HS 20 live load moment/plank. For wheel load distribution to deck planks, dist. normal to traffic = 1 wheel/plank, dist. in direction of traffic = tire width (assume 20" wide tire). Therefore the moment for a uniform load spread over 20" at center of span length "S" and using a continuous factor of ".8" is: Mom = .8(P/2 x S/2 - P/2 x 5/12) = .8P(S/4 - .2083). For HS20 loading, P = 16 kips, Legal P = 9 kip (18/2). Therefore $M_{(HS)}$ = .8 x 16(S/4 -.2083) = 12.8(S/4 - .2083) & M(legal) = 7.2(S/4 - .2083) = .5625 $M_{HS}$. POSTING : M(legal) = .5625 $M_{(HS)}$. See explanation above.**

## TABLE OF RATING & POSTING LOADS FOR 3" x 12" TRANS. TIMBER PLANK ASSUME fi = 1.5 ksi, fo = 2.0 ksi

| C/C | Spacing "S" feet | $M_{HS}$ ft-kips | HS Inv. Rating = 2.24 (20) / $M_{HS}$ | HS Oper Rating = 2.99 (20) / $M_{HS}$ | $M_{LEGAL}$ ft-kips | POSTING--TONS | |
|---|---|---|---|---|---|---|---|
| | | | | | | $M_3$ = 2.99 (24) / $M_{LEG}$ | $M_{3S2}$ = 2.99(36.6) / $M_{LEG}$ |
| 2.0 | 1.77 | 3.0 | HS 14.9 | HS 19.9 | 1.69 | LEGAL | LEGAL |
| 2.25 | 2.02 | 3.8 | 11.8 | 15.7 | 2.13 | LEGAL | LEGAL |
| 2.5 | 2.27 | 4.6 | 9.7 | 13.0 | 2.59 | LEGAL | LEGAL |
| 2.75 | 2.52 | 5.4 | 8.3 | 11.0 | 3.04 | LEGAL | LEGAL |
| → 3.0 | 2.77 | 6.2 | 7.2 | 9.6 | 3.49 | 20 | 31 |
| 3.25 | 3.02 | 7.0 | 6.4 | 8.5 | 3.94 | 18 | 28 |

2-20-98

STATE OF MINNESOTA
DEPARTMENT OF TRANSPORTATION

Sheet No. ____________
Rated by ____________
Checked by ____________
Date ____________
Side 1

RATING SHEET FOR
TRANSVERSE TIMBER PLANK DECK

## BRIDGE LOCATION AND DESCRIPTION

Bridge No.____________
Route ____________
Year Built ____________
Year Remodeled ____________

Long. Beam Spacing ( C/C)____________
Transverse Plank Thickness (t) ____________
Transverse Plank Width (b) ____________

## SUMMARY OF RATING AND LOAD POSTING

| Inventory rating | Operating Rating | Load Posting Req'd? | LOAD POSTING LIMITS @ (Complete when load posting is required) | | |
|---|---|---|---|---|---|
| | | | Vehicle Type M3 Weight = 24T | Semi-trailer Comb. Type M3S2 Weight = 36T | Truck & Full Trailer Type M3S3 Weight = 40T |
| H S______ | HS ______ | ______Yes<br>______No | ________Tons | ________Tons | ________Tons |

## RATING FROM WORK SHEET TABLE OR CALCULATIONS BELOW

(See back side for explanation)

S = C/C - 0.23 = ____________(ft) No Impact

| | | INVENTORY | OPERATING |
|---|---|---|---|
| Allowable Stress (assumed) | kips/sq.in | $f_i = 1.50$ | $f_o = 1.33\, fi = 2.0$ |
| Mom Avail. For LL/Plank = SM(fi or fo)/12 -.01 | Ft. Kips | ① | ① |
| M (HS) = 12.8 ( S/4 - .2083 ) | Ft. Kips | ② | ② |
| RATING = $\frac{①}{②}$ (20) | number | HS | HS |

**POSTING:** M ( LEGAL) = .5625M (HS) = ____________

Posting Limits:

Vehicle (M3) = $\frac{\text{Mom Avail(Oper)}}{\text{M (legal)}}$ (24) = ____________tons

Combination Vehicles (M3S2 or M3S3 ) = $\frac{\text{Mom Avail(Oper)}}{\text{M( legal)}}$ (36.65) = ____________Tons

## WORK SHEET EXPLANATION

**S = C/C - 0.23: See AASHTO Sect. 3.25.1.2. Assume flange width = 5.5", then S = C/C - 5.5/2x12 = C/C - 0.23.**

**Allowable Stress: For older timber planks where timber stress is unknown, use fi = 1.5 ksi & fo = 2.0 ksi. If timber bending stress is known, see AASHTO 13.6.4.1. For 2" to 4" thick planks, Fb' = Fb,$C_M$,$C_D$,$C_{fu}$ and if $C_M$ =0.85, $C_D$ = 1.15, $C_{fu}$ = 1.2, then Fb' = Fb (1.17). Example: 3" x 12" plank, Fb (plan) = 1.4 ksi, then fi = 1.4(1.17) = 1.64 ksi and fo = 1.33 x 1.64 = 2.18 ksi.**

**Moment available for LL/plank: Assume dead load moment of 0.01 ft-kip, then Mom avail. = SM(fi or fo)/12 - .01 where SM = (b)(t)$^2$/6. Example: 3" x 12" plank, fi = 1.5 ksi, then SM = (12 x 3 x 3)/6 = 18 in cu. and Mom avail. (Inv) = 18 x 1.5/12 - .01 = 2.24 ft kip/plank.**

**M ($_{HS}$) = HS 20 live load moment/plank. For wheel load distribution to deck planks, dist. normal to traffic = 1 wheel/plank, dist. in direction of traffic = tire width (assume 20" wide tire). Therefore the moment for a uniform load spread over 20" at center of span length "S" and using a continuous factor of ".8" is: Mom = .8(P/2 x S/2 - P/2 x 5/12) = .8P(S/4 - .2083). For HS20 loading, P = 16 kips, Legal P = 9 kip (18/2). Therefore M($_{HS}$) = .8 x 16(S/4 -.2083) = 12.8(S/4 - .2083) & M(legal) = 7.2(S/4 - .2083) = .5625 $M_{HS}$. POSTING : M(legal) = .5625 M($_{HS}$). See explanation above.**

TABLE OF RATING & POSTING LOADS FOR 3" x 12" TRANS. TIMBER PLANK
ASSUME fi = 1.5 ksi, fo = 2.0 ksi

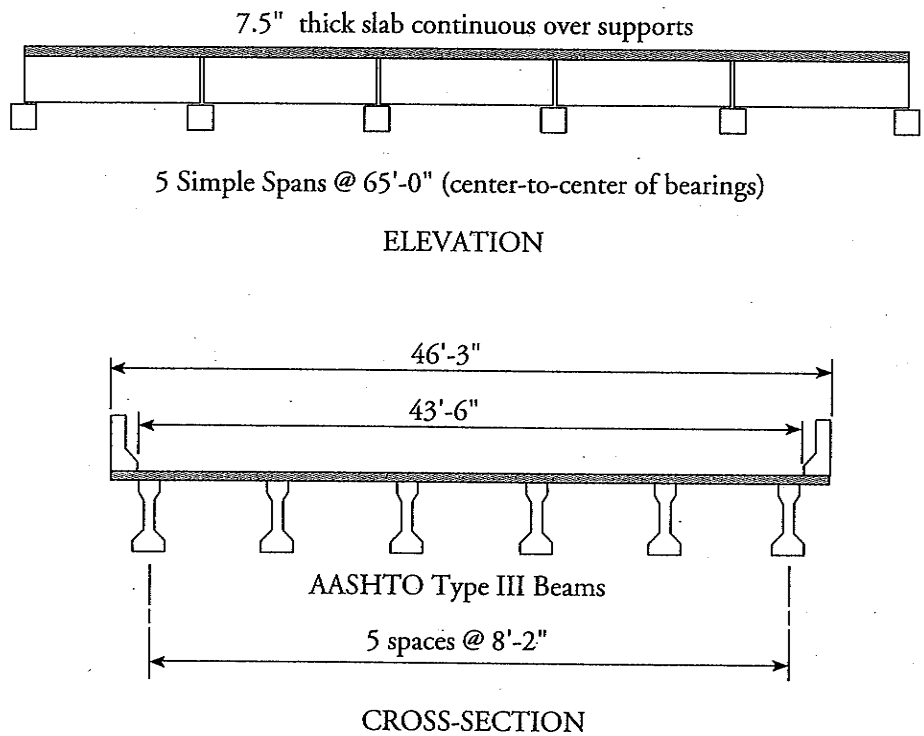
| C/C | Spacing "S" feet | $M_{HS}$ ft-kips | HS Inv. Rating = 2.24 (20) / $M_{HS}$ | HS Oper Rating = 2.99 (20) / $M_{HS}$ | $M_{LEGAL}$ ft-kips | POSTING--TONS $M_3$ = 2.99 (24) / $M_{LEG}$ | POSTING--TONS $M_{3S2}$ = 2.99(36.6) / $M_{LEG}$ |
|---|---|---|---|---|---|---|---|
| 2.0 | 1.77 | 3.0 | HS 14.9 | HS 19.9 | 1.69 | LEGAL | LEGAL |
| 2.25 | 2.02 | 3.8 | 11.8 | 15.7 | 2.13 | LEGAL | LEGAL |
| 2.5 | 2.27 | 4.6 | 9.7 | 13.0 | 2.59 | LEGAL | LEGAL |
| 2.75 | 2.52 | 5.4 | 8.3 | 11.0 | 3.04 | LEGAL | LEGAL |
| 3.0 | 2.77 | 6.2 | 7.2 | 9.6 | 3.49 | 20 | 31 |
| 3.25 | 3.02 | 7.0 | 6.4 | 8.5 | 3.94 | 18 | 28 |

## 18.5 RATING EXAMPLE

### 18.5.1 Introduction

This two-lane bridge, built in the 1970s, is located on State Road 30 over the Carrabelle River in Franklin County, FL. It consists of five simple spans, each 65-ft. long. Each bridge span consists of 6 AASHTO Type III prestressed concrete beams spaced at 8'-2" on center. The total width of the bridge is 46'-3". The bridge has an 8-in.-thick continuous concrete deck. The top 1/2 in. of the slab is considered a wearing surface. The continuity of slab is not considered in the following calculation for simplicity. **Figure 18.5.1-1** shows the bridge elevation and the typical span cross-section. The following calculations demonstrate the rating procedure for an interior beam using AASHTO Specifications and the field test method.

*Figure 18.5.1-1*
*Example Bridge Details*




### 18.5.2 Materials and Conditions

Number of simple spans = 5

Number of traffic lanes = 2

Span length, L = 65 ft

Bridge width = 46'-3"

Structural slab thickness, $t_s$ = 7.5 in.

Total slab thickness = 8 in.

Future wearing surface = 2.0 in. (25 psf)

Parapet weight = 411 plf

Specified concrete strength of beam, $f'_c$ = 5,000 psi

Modulus of elasticity of beam concrete, $E_c$ = 4,287 ksi

Specified Concrete strength at transfer (beam), $f'_{ci}$ = 4,000 psi

Modulus of elasticity of beam concrete at transfer, $E_{ci}$ = 3,834 ksi

Specified concrete strength of deck, $f'_c$ = 3,400 psi

Modulus of elasticity of deck concrete, $E_c = 3,535$ ksi

Unit weight of concrete for beams and deck, $w_c = 150$ pcf

Allowable tensile stress at service (midspan) $= -6\sqrt{f'_c} = -0.424$ ksi

Prestressing strand strength, $f'_s = 270$ ksi

Modulus of elasticity of strand, $E_s = 28,500$ ksi

Area of 1/2 in. dia prestressing strand = 0.153 in.$^2$

Initial prestress, $f_{si} = 0.75f'_s = 202,500$ psi

Initial prestress force/strand, $P_j = (0.153)(0.75)(f'_s) = 30.98$ kips

Rating vehicle = HS20

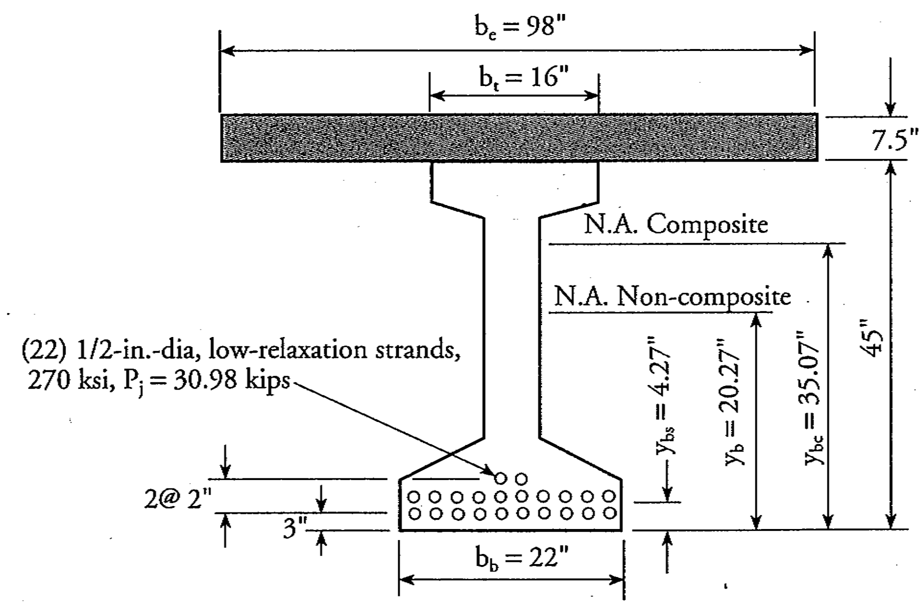
### 18.5.3 Section Properties

The beam properties and cross-section are shown in **Table 18.5.3-1** and **Fig. 18.5.3-1.** The section properties are calculated based on a 7.5 in. structural slab thickness, $t_s$. The difference in material properties between slab and beam are considered with transformed width of slab.

*Table 18.5.3-1 Section Properties*

| Non-Composite Section | Composite Section |
|---|---|
| $y_t = 24.73$ in. | $y_{tc} = 17.42$ in. |
| $y_b = 20.27$ in. | $y_{bc} = 35.08$ in. |
| $I = 125,390$ in.$^4$ | $I_c = 364,153$ in.$^4$ |
| $A = 560$ in.$^2$ | $A_c = 1,166$ in.$^2$ |

*Figure 18.5.3-1 Cross-Section at Midspan*




### 18.5.4 Load Calculations

#### 18.5.4.1 Dead Loads

The non-composite section carries the beam self-weight and slab weight (8-in. thick), while the weights of the barrier and future wearing surface are uniformly distributed among the six beams and are carried by the composite section.

Beam moment:

$$M_g = \frac{wL^2}{8} = \frac{(560/144)(0.150)(65)^2}{(8)} = 308.07 \text{ ft-kips}$$

Slab moment:

$$M_s = \frac{wL^2}{8} = \frac{(8.17)(8/12)(0.150)(65)^2}{(8)} = 431.30 \text{ ft-kips}$$

Barrier moment:

$$M_b = \frac{wL^2}{8} = \frac{(0.411)(2)(65)^2}{(8)(6)} = 72.35 \text{ ft-kips}$$

Future wearing surface:

$$M_{ws} = \frac{wL^2}{8} = \frac{(43.5)(0.025)(65)^2}{(8)(6)} = 95.72 \text{ ft-kips}$$

Total dead load moment:

$M_d = 907.62$ ft-kips

**18.5.4.2 Live Loading (HS20 Truck)**

Truck loading governs for this span, so lane and alternate military loadings are not considered.

Maximum wheel-load moment:

$M_{WL\text{-}HS20} = 448$ ft-kips

Impact factor:

$$I = \frac{50}{(125+65)} = 0.26$$ [STD Eq. 3-1]

AASHTO wheel-load distribution factor:

$$WDF = \frac{S}{5.5} = \frac{8.17}{5.5} = 1.49$$ [STD Table 3.23.1]

Live load moment per beam:

$M_{LL+I} = (WDF)(M_{WL\text{-}HS20})(1 + I) = (1.49)(448)(1.26) = 841.1$ ft-kips

**18.5.4.3 Prestress Losses**

Initial prestress force immediately after transfer: [STD Art. 9.16.2.1.2]

$P_{si} = (22)(0.153)(0.69)(270) = 627.1$ kips

Eccentricity of prestress force:

$e = y_b - y_{bs} = 20.27 - 4.27 = 16.00$ in. [STD Art. 9.16.2.1.2]

$$f_{cir} = \frac{P_{si}}{A} + \frac{P_{si}e^2}{I} - \frac{M_g e}{I}$$

$$= \left(\frac{627.1}{560}\right) + \left(\frac{(627.1)(16)^2}{125{,}390}\right) - \left(\frac{(308.07)16(12)}{125{,}390}\right) = +1.93 \text{ ksi}$$

$$f_{cds} = -\frac{M_s e}{I} - \frac{(M_b + M_{ws})(y_{bc} - y_{bs})}{I_c}$$

$$= -\left(\frac{(431.48)16(12)}{125{,}390}\right) - \frac{(72.35 + 95.72)(12)(35.07 - 4.27)}{364{,}324} = -0.831 \text{ ksi}$$

Elastic shortening loss:

$$ES = \frac{E_s}{E_{ci}} f_{cir} = \left(\frac{28{,}500}{3{,}834}\right)(1.93) = 14.35 \text{ ksi}$$ [STD Eq. 9-6]

Shrinkage loss (assume RH = 70%):

$SH = 17 - (0.15)(RH) = 17 - (0.15)(70) = 6.50$ ksi [STD Eq. 9-4]

Creep loss:

$CR_c = 12f_{cir} - 7f_{cds} = (12)(1.93) - (7)(0.831) = 17.34$ ksi [STD Eq. 9-9]

Relaxation loss:

$CR_s = 5 - 0.1(ES) - 0.05(SH + CR_c) = 5 - (0.1)(14.35) - (0.05)(6.50 + 17.34)$
$= 2.37$ ksi [STD Eq. 9-10]

Total prestress losses:

$SH + ES + CR_c + CR_s = 6.50 + 14.35 + 17.34 + 2.37 = 40.56$ ksi [STD Eq. 9-3]

Effective final prestress:

$f_{se} = 202.5 - 40.56 = 161.94$ ksi

Effective final prestress force:

$P_{se} = (22)(0.153)(161.94) = 545.09$ kips

### 18.5.5 Stresses and Strength

In a complete design process, strength checking (bending and shear) should be conducted for several sections along the span length. While a rating process should follow the same principles as design, the following calculation is limited to the bending strength at midspan and stress at the bottom of the beam, which are the locations that typically govern design.

**18.5.5.1 Total Tensile Stress at Service - Inventory Rating**

Dead load stress on non-composite section:

$$f_{NDL} = -\frac{(M_g + M_s)y_b}{I} = -\frac{(308.07 + 431.48)(12)(20.27)}{125,390} = -1.435 \text{ ksi}$$

Dead load stress on composite section:

$$f_{CDL} = -\frac{(M_b + M_{ws})y_{bc}}{I_c} = -\frac{(72.35 + 95.72)(12)(35.07)}{364,324} = -0.194 \text{ ksi}$$

Live load stress on composite section:

$$f_{LL+I} = -\frac{(M_{LL+I})y_{bc}}{I_c} = -\frac{(841.1)(12)(35.07)}{364,324} = -0.972 \text{ ksi}$$

Stress from prestress force:

$$f_{pe} = \frac{P_{se}}{A} + \frac{P_{se}ey_b}{I} = \frac{545.09}{560} + \frac{(545.09)(16)(20.27)}{125,390}$$

$$= +2.383 \text{ ksi (compression)}$$

Total tensile stress at service:

$$f_{total} = f_{pe} - (f_{NDL} + f_{CDL}) - f_{LL+I}$$

$$= 2.383 - (1.435 + 0.194) - 0.972 = -0.218 \text{ ksi,}$$

$$f_{allow} = -0.424 \text{ ksi}$$

$$R.F._{IN} = f_{allow}/f_{total} = 0.424/0.218 = 1.94$$

**18.5.5.2 Design Flexural Strength - Operating Rating**

$$M_u = 1.3(M_d + 1.67M_{LL+I}) = 1.3[(907.62) + 1.67(841.1)] = 3,005.93 \text{ ft-kips}$$

$$d = 45 + 7.5 - 4.27 = 48.23 \text{ in.}$$

$$f^*_{su} = f'_s\left[1 - \frac{\gamma^*}{\beta_1}\left(\frac{p^*f'_s}{f'_c}\right)\right]$$

$$= 270\left[1 - \left(\frac{0.28}{0.85}\right)\frac{(0.153)(22)}{(98)(48.23)}\left(\frac{270}{3.4}\right)\right] = 265.0 \text{ ksi}$$

$$a = \frac{A^*_s f^*_{su}}{0.85bf'_c} = \frac{(265.0)(22)(0.153)}{(0.85)(98)(3.4)} = 3.15 \text{in.}$$

$$M_n = A^*_s f^*_{su}\left(d - \frac{a}{2}\right) = (22)(0.153)(265.0)\left(48.23 - \frac{3.15}{2}\right)\left(\frac{1}{12}\right) = 3,468.0 \text{ ft-kips}$$

$$R.F._{OP} = \frac{M_n}{M_u} = \frac{3,468.0}{3,005.93} = 1.15$$

**18.5.5.3 Rating Factor**

The above calculations show that the design is governed by the lower rating factor for the design flexural strength ($R.F._{OP}$ = 1.15). Therefore, the load rating for the bridge is equal to HS(20 x 1.15) = HS23.

**18.5.6 Evaluation Guide Specification Rating**

Based on values obtained using the *Standard Specifications* in the previous section, the following ratings can be obtained from the *Evaluation Guide Specifications.*

Operating and inventory ratings using the factored load method:

$$R.F._{OP} = \frac{\phi M_n - 1.3 M_d}{1.3\ M_{LL+I}} = \frac{(1.0)(3{,}468.0) - (1.3)(907.62)}{(1.3)(841.1)} = 2.09$$

$$R.F._{IN} = \frac{\phi M_n - 1.3 M_d}{1.3(1.67) M_{LL+I}} = \frac{(1.0)(3{,}468.0) - (1.3)(907.62)}{(1.3)(1.67)(841.1)} = 1.25$$

Inventory rating using the allowable stress method:

$$R.F._{IN} = \frac{f_{pe} - f_{DL} - f_{allow}}{f_{LL+I}} = \frac{2.383 - (1.435 + 0.194) - (-0.424)}{0.972} = 1.21$$

The inventory load rating is controlled by the service (allowable stress) requirement. Therefore, the inventory rating is equal to 1.21. The inventory rating with the factored load method is 1.25, which shows that the use of this method may result in a higher rating. The final rating factor for prestressed concrete structures should be the lesser of the values obtained by the allowable stress and factored load methods to ensure adherence to the original design assumptions.

**18.5.7 Rating using the LRFD Specifications**

As noted in Section 18.1, at present there are no established guidelines for load rating using the *LRFD Specifications.* The following rating is based on the guidelines given in the *Standard Specifications* and is intended to illustrate the difference between the two design specifications.

**18.5.7.1 Dead Load**

The dead loads are essentially the same as previously calculated.

**18.5.7.2 Prestress Loss**

The prestress loss calculation in the *LRFD Specifications* is almost identical to that of the *Standard Specifications* except for minor changes to the relaxation calculation. The total loss calculated according to the *LRFD Specifications* is 41.62 ksi compared to 40.56 ksi using the *Standard Specifications.* Since these values are fairly close, the effective prestress will be taken as the same value above, computed using the *Standard Specifications*, i.e., 161.94 ksi.

**18.5.7.3 Live Load**

Rating live load: HL-93

Maximum truck moment per lane: $M_{lane\text{-}HS20(truck)}$ = 896.0 ft-kips

Maximum lane moment per lane: $M_{lane\text{-}HS20(lane)}$ = 338.0 ft-kips

Maximum tandem moment per lane: $M_{lane\text{-}HS20(tandem)}$ = 762.5 ft-kips

Dynamic load allowance: IM = 0.33

AASHTO lane-load distribution factor for cross-section "type k" with two or more lanes loaded:

$$LDF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0 L t_s^3}\right)^{0.1} \qquad \text{[LRFD Table 4.6.2.2.2b-1]}$$

where

$$K_g = n(I + Ae_g^2) \quad \text{[LRFD Eq. 4.6.2.2.1-1]}$$

$$= \frac{4,287}{3,535}\left(125,390 + 560\left(45 + 7.5 - 20.27 - \frac{7.5}{2}\right)^2\right) = 702,913 \text{ in.}^4$$

$$LDF = 0.075 + \left(\frac{8.17}{9.5}\right)^{0.6}\left(\frac{8.17}{65}\right)^{0.2}\left(\frac{702,913}{(12.0)(65)(7.5)^3}\right)^{0.1} = 0.726$$

The live load moment is specified as the lane load moment plus the larger of the truck or the tandem moment:

$$M_{LL+I} = LDF[M_{lane-HS20(lane)} + \max(M_{lane-HS20(truck)}, M_{lane-HS20(tandem)})(1 + IM)]$$
$$= (0.726)[338.0 + 896.0(1.33)] = 1,110.5 \text{ ft-kips}$$

**18.5.7.4 Live Load Stress**

$$f_{LL+I} = \frac{(M_{LL+I})y_{bc}}{I_c} = \frac{(1,110.5)(12)(35.07)}{364,324} = 1.280 \text{ ksi}$$

It can be seen that the live load stress calculated by the *LRFD Specifications* is much higher than that calculated by the *Standard Specifications* (1.280 ksi versus 0.972 ksi).

**18.5.7.5 Strength Calculation**

Because the strength calculation in the *LRFD Specifications* is similar to that in the *Standard Specifications*, the detailed calculations are not presented here. For a presentation of the details, consult Section 8.2. To review a sample calculation, refer to Design Example 9.4.

$$M_n = A_{ps}f_{ps}\left(d - \frac{a}{2}\right) = (22)(0.153)(264.21)\left(48.23 - \frac{3.14}{2}\right)\left(\frac{1}{12}\right) = 3,458.0 \text{ ft-kips}$$

This is very close to the value obtained using the *Standard Specifications* (3,468.0 ft-kips). The minor variation in the nominal moment capacity is due to the change in the calculation for "$f_{ps}$" and sometimes for the calculation for "a" in the *LRFD Specifications*.

**18.5.7.6 Inventory Rating**

The inventory rating should utilize the same principles that were used in the design. Therefore, the inventory rating, according to Strength I, should be:

$$R.F._{IN} = \frac{\phi M_n - 1.25(M_g + M_s + M_b) - 1.5M_{ws}}{1.75M_{LL+I}}$$

$$= \frac{(1.0)(3,458.0) - (1.25)(308.07 + 431.48 + 72.35) - (1.5)(95.72)}{(1.75)(1,110.5)} = 1.18$$

Although the live load moment by the *LRFD Specifications* analysis is 32 percent larger than that by the *Standard Specifications* (1,110.5 ft-kips versus 841.1 ft-kips), the load factor for live load is considerably less for the *LRFD Specifications* analysis (1.75 versus 2.17). Thus the inventory rating for *LRFD Specifications* Strength I is only 6 percent less than that for the *Standard Specifications* (1.18 versus 1.25).

Inventory rating with *LRFD Specifications* Service III (full dead load plus 80 percent live load):

Allowable tensile stress:

$$R.F._{IN} = \frac{f_{pe} - f_{DL} - f_{allow}}{0.8 f_{LL+I}} = \frac{2.383 - (1.435 + 0.194) - (-0.424)}{(0.8)(1.28)} = 1.15$$

Theoretically, the rating should also be calculated by checking compressive stress. This is not presented here because it is not critical in this example. The final rating should be the lesser of the service and strength check.

The procedures for determining operating rating with the *LRFD Specifications* have not yet been adopted. Therefore, it is recommended that the operating rating be taken as 1.67 times the inventory rating, the same factor used in the *Standard Specifications*.

## 18.5.8 Rating by Load Testing

### 18.5.8.1 Test Information

Target inventory rating: HS20

Maximum test load = two test trucks, each with a gross weight of 207 kips

Maximum moment per test truck per lane = 1,868 ft-kips

Solving for the live load stress that will cause the bottom fiber stress to reach the allowable stress:

$f_{LL} = f_{pe} - f_{DL} - f_{allow} = 2.383 - (1.435 + 0.194) - (-0.424) = 1.178$ ksi

The stress, $f_{LL}$, is theoretically the maximum stress that the bridge can be subjected to without cracking. Since stress cannot be measured directly, the strain corresponding to the stress, $f_{LL}$, is computed as: $\varepsilon_{LL} = f_{LL}/E_c = 1.178/4{,}287 = 0.000275 = 275 \times 10^{-6}$. This strain value (275 microstrains) is considered the limiting test strain to avoid any permanent damage to the bridge during testing.

### 18.5.8.2 Test Results

In this example, the bridge was loaded incrementally using the two test trucks and the strains were monitored at all critical locations. The maximum measured strain at the bottom of the beam under the two test vehicles was 182 microstrains, which is 66 percent of the calculated strain limit, $\varepsilon_{LL}$. The bridge showed no signs of distress or cracking at any load level and load-strain relationships were linear throughout the test.

### 18.5.8.3 Distribution Factor

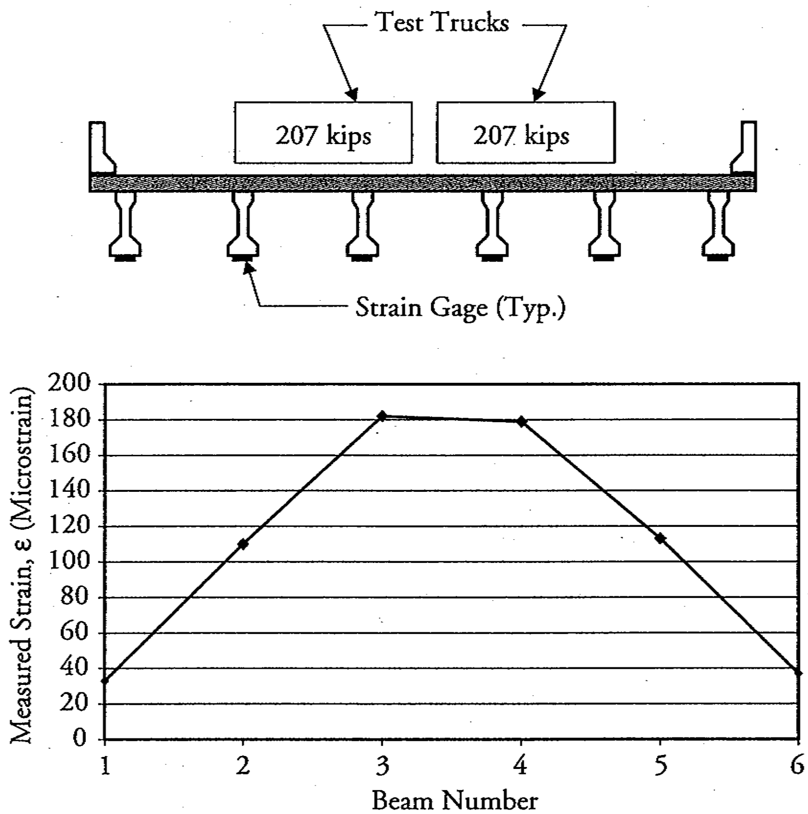
The strain measurements across the bridge under maximum applied live loads are shown in **Fig. 18.5.8.3-1**.

Calculate the measured wheel load distribution factor, WDF:

$$WDF = \frac{N_w \varepsilon_{max}}{\Sigma \varepsilon_i} = \frac{(4)(182)}{33 + 110 + 182 + 179 + 113 + 37} = 1.11$$

where $N_w$ is the number of lines of wheel loads on the tested bridge.

*Fig. 18.5.8.3-1*
***Strain Distribution in Bottom Flanges across Bridge***




For comparison, the wheel load distribution factor from the *Standard Specification* is 1.49. The LRFD lane distribution factor calculated in Section 18.5.7.3 must be multiplied by 2 for comparison, which is (2)(0.726) = 1.45. Therefore, the distribution factor determined by the load test is much lower than the values computed by the two specifications.

Because the test vehicles are different from the HS20 truck, the stress from the test for an equivalent HS20 truck plus impact can be calculated from the ratio of test truck moment and HS20 moment as:

$$f_{HS20+I} = \left(\frac{(1+I)M_{WL-HS20}}{M_{test}}\right)(-\varepsilon_{measured})(E_c) = \left(\frac{(2)(1.26)(448)}{1,868}\right)(-182\times10^{-6})(4,287)$$
$$= -0.471 \text{ ksi}$$

where the factor 2 equates the maximum wheel load moment for an HS20 truck to two test trucks.

Because the applied test load moment per lane (1,868 ft-kips/lane) is less than the ultimate design live load moment per lane [2.17(2)(448)(1.26) = 2,450 ft-kips] the load test is considered diagnostic.

**18.5.8.4 Test Inventory Rating Factor**

The inventory load rating based on the test measurements is shown below.

$$R.F._{IN} = \frac{f_{pe} - f_{DL} - f_{allow}}{f_{HS20+I}} = \frac{2.383 - (1.435 + 0.194) - (-0.424)}{0.471} = 2.50$$

The above test inventory rating, based on test measurements, is more than twice the theoretical AASHTO allowable stress inventory rating obtained in Section 18.5.6. The computed maximum tensile stress from load testing, 0.471 ksi, is significantly less than the theoretically calculated value, 0.972 ksi. This is due to many beneficial factors that are ignored in a theoretical load rating. These factors include:

- slab continuity
- diaphragms
- parapet composite action
- bearing restraint effects
- lower than expected prestress losses
- higher concrete strength
- higher concrete modulus of elasticity

In addition, the AASHTO load distribution factors are generally very conservative resulting in the design of stronger elements than required by actual loading.

**18.5.8.5 Test Operating Rating Factor**

Maximum live load moment from test for equivalent HS20 plus impact:

$$M_{LL+I} = \left(\frac{I_c}{y_{bc}}\right) f_{HS20+I} = \left(\frac{364{,}324}{35.07}\right)\left(\frac{0.471}{12}\right) = 407.7 \text{ ft-kips}$$

Operating rating according to the *Standard Specifications*, Load Factor Method:

$$R.F._{OP} = \frac{\phi M_n - 1.3M_d}{1.3M_{LL+I}} = \frac{1.0(3{,}458.0) - 1.3(907.62)}{1.3(407.7)} = 4.30$$

The inventory rating factor (2.50) and operating rating factor (4.30) above are considered upper bounds due to the nature of diagnostic/linear analysis. Therefore, the final rating should be limited to the original design, i.e., inventory rating of HS20, or operating rating of HS(20 x 1.67) = HS33.

**18.6 REFERENCES**

*Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges*, American Association of State Highway and Transportation Officials, Washington, DC, 1989

*Manual for Condition Evaluation of Bridges*, American Association of State Highway and Transportation Officials, Washington, D.C, 1994

*AASHTO LRFD Bridge Design Specifications*, Second Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1998

*Standard Specification for Highway Bridges*, 16th Edition, American Association of State Highway and Transportation Officials, Washington, DC, 1996